FINAL GEOTECHNICAL REPORT

Chehalis Western Trail Pedestrian Bridge

SR-5, XL-2315, MP 108.2

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1. Project Overview

A new pedestrian crossing is being planned along the old Chehalis Western Railroad grade approximately 0.26 miles east of the junction of I-5 and Sleater Kinney Road in Lacey. The bridge will provide foot traffic access over I-5, existing I-5 Trail, and an existing south- bound on-ramp from Sleater Kinney to I-5. The new bridge will be an integral part of the Chehalis Western Trail. The work includes up to 6 new retaining walls, and a two span bridge.

2. Regional Setting

2.1. Surface and Topographic Conditions

Land use along the project alignment is in a commercial/industrial area and is located west of South Sound Shopping Center. The alignment is relatively flat and dominated by one to two-story office buildings and commercial buildings, and associated parking lots.

2.2. Regional Geology

The project site is located in the lower portion of the Puget Sound lowland between the Cascade Range to the east and the Olympic Mountains to the west. The present day topography and the near-surface geology of the Puget Lowland are largely the result of multiple cycles of continental glaciation that occurred during the Pleistocene epoch. The last major episode was the Frasier glaciation, which occurred roughly between 20,000 and 12,000 years ago. The Frasier ice sheet is believed to have filled the lowland to a thickness of up to one mile in the deepest part of the trough. As the glacier retreated, a variable thickness of glacial sediment was deposited.

The Puget Sound area is primarily underlain by a thick, complex sequence of glacial and interglacial sediments. Meltwater flowing from the advancing ice sheet transported a variety of sediment that built a broad outwash plain. Coarse sediment such as sand and gravel was deposited in the high-energy environment near the advancing and recessing glacier. Finer sediments such as silt and clay were deposited in a low-energy environment further from the glacier, in ponds and lakes. These ponds and lakes were formed as advancing ice sheet blocked regional drainages. As the ice sheet advanced, the previously deposited sediments were overridden, compacting the deposits to their present dense condition. Following the last glacial advance and retreat, alluvial (river) and lacustrine (lakebed) sediments were deposited by runoff from the slopes of the Olympic and Cascade Ranges, and the melt waters of the glaciers. The more recent portions of these sediments (lower-energy) consist of fine-grained sands, silts, and clay.

As part of this study, we reviewed the available geologic data in the vicinity of the project. Washington Division of Geology and Earth Resources Open File Report 87-3, *Geologic Map of the Souther Half of the Tacoma Quadrangle, Washington* by Timothy J. Walsh, 1987, indicates the site lies in Vashon Recessional Outwash or Vashon Till.

Vashon Recessional Outwash deposits (Qdvs, Upper Pleistocene) are stream deposited unconsolidated glacial and pro-glacial deposits, which consist of loose to dense silty sand with gravels.

Vashon Till deposits (Qdvt, Upper Pleistocene) were mapped east of the project. These soils consist of over-consolidated glacial deposits, which consist of a dense to very dense mixture of clay, silt, sand, gravel, cobbles and boulders deposited directly by glacier ice. Local deposits of advance outwash sand and gravels are contained both within and overlying the till.

2.3. *Regional Seismicity*

Washington is situated at a convergent continental margin, the collision boundary between two tectonic plates. The Cascadia subduction zone, which is the convergent boundary between the North American plate and the Juan de Fuca plate, lies offshore from northernmost California to southernmost British Columbia. The northward-moving Pacific plate is pushing on the Juan de Fuca plate, causing complex seismic strain to accumulate. Earthquakes are caused by the abrupt release of this slowly accumulated strain.

There are three types or sources of earthquakes in Washington. The first is Intraplate or Benioff Zone Earthquakes. Intraplate or Benioff earthquakes occur in the subducting Juan de Fuca plate at depths of 25–100 km (15–62 miles). The largest of these recorded were the magnitude (M) 7.1 Olympia Earthquake in 1949, the M 6.5 Seattle-Tacoma earthquake in 1965, M 5.1 Satsop earthquake, and the M 6.8 Nisqually earthquake of 2001.

The second source or type of earthquake is the Shallow Crustal Earthquake. Shallow crustal earthquakes occur within 24 to 30 km (15 to 19 miles) of the surface. The most recent example was an M 3.5 earthquake approximately 7 miles east of the site on June 19, 2003. The majority of the shallow earthquakes are less than M 4.0 in the vicinity of the site. Significant earthquakes (magnitude greater than 4.0) occurring on approximately 20-mile radius of the site include the following events; a magnitude 5.4 earthquake occurred approximately 5 miles ENE of Duvall (13 miles NE of the site) on May 3, 1996; a magnitude 4.2 earthquake occurred 13 miles ESE of the site on December 31, 1978; a magnitude 4.1 earthquake occurred approximately 9 miles SSW of the site on December 28, 1971; a magnitude 4.0 earthquake occurred approximately 2.5 miles NNE of the site on July 30, 1964; a magnitude 4.6 earthquake occurred approximately 18 miles ESE of the site on January 24, 1963; and, on August 6, 1932, a M 5.0 earthquake occurred approximately 8 miles to the WNW of the site.

The third source or type of earthquake is the Subduction Zone (Interplate) Earthquake. Subduction zone earthquakes occur along the interface between tectonic plates. Compelling evidence for great-magnitude earthquakes along the Cascadia subduction zone has recently been discovered. These earthquakes were evidently enormous (M 8-9+) and recurred on average every 550 years. The last of these great earthquakes struck Washington about 300 years ago.

The Puget Lowland is believed to have a series of buried geologic structures. The east-west trending Seattle fault is approximately 45 miles NE of the site. Two northeast-trending gravity/magnetic anomalies are observed, one beneath the site and the other is approximately 24 mile away in the vicinity of Comment Bay in Tacoma. No recorded movement along these anomalies has been observed. The Seattle fault has know to have moved approximately 1000 years ago and has generated massive landslides into Lake Washington.

3. Site Investigation

3.1. Previous Studies

A series of field investigations have been conducted in the vicinity of this site. Specifically, the original design investigation was conducted for a railroad bridge constructed in 1956, and a proposed replacement railroad bridge designed in 1984. The replacement bridge was never constructed and the 1956 bridge was demolished in the late 1980's. In the process of demolition the existing center pier was left in place. Four test holes were conducted in these previous investigations. The information, which was used in this report, is listed below:

- Two test holes drilled east of the proposed alignment in 1956.
- Two test holes drilled for the foundation of a proposed bridge west of the existing alignment in 1984.

The boring logs are provided in Appendix B, and their locations are shown on Figure A-2 in Appendix A.

3.2. Exploration Program

The most recent investigation was designed to provide additional subsurface information to better define the foundation conditions for the final design of the proposed pedestrian structure. A total of 3 additional test holes were drilled. The current field investigation included the following information:

- Two test holes drilled at the abutment piers
- One test hole drilled for the interior pier.

The test hole logs are provided in Appendix B, and their locations are shown on Figure A-2 in Appendix A. The edited logs of the test boring should be included in the final contract documents.

4. Laboratory Testing

Laboratory testing was performed on selected samples from the field exploration program. The testing consisted of performing particle size analyses, determining the liquid limit if applicable, and determining the plastic limit and plasticity index, if applicable. The tests were done in accordance with AASHTO T-88, T-89, and T-90 guide specifications respectively. After the testing was completed, the samples were classified using the Unified Soil Classification System (USCS).

The results from this and previous laboratory testing were used to establish geotechnical design parameters. The results of all laboratory testing are summarized in Appendix C.

5. Site Conditions

5.1. Soil Conditions

In general, the soils encountered were grouped into engineering units based on similarities in materials and engineering properties. In Appendix A, soil profiles were developed along the project alignment to show the various soil types relative to the planned structures on this project. The principle units along the alignment can be summarized as follows:

- **Unit 1 –Fill and Recessional Outwash** - Loose to dense, silty sand with gravel. The thicknesss of this unit varies up to 10 ft in places along the alignment.
- **Unit 2 - Advance Outwash and Vashon Till** – These deposits consist of dense to very dense, poorly graded gravel with sand and silt to well graded gravel with sand, cobbles and boulders.

These soil units are consistent with the Geologic History of the site and are greatly influenced by the glacial activity that occurred in the region.

5.2. Surface Water and Ground Water

The ground water varies with the topography and geologic soil units along the project alignment. In general, ground water varies seasonally between the wet winter and spring months and the dryer summer and early fall months. The highest ground water observations occurred between December and April at this site. The lowest readings occurred in summer and late fall. Water levels are summarized in the test hole logs. An open-stand pipe piezometer was installed in H-2-05 to measure the seasonal variations in the water level readings.

The water levels were observed to vary between elevations 195.4 ft at H-1-84 and 189.5 ft at H-2-04 at the time of drilling. In general, ground water should be below the foundations for Pier 1 and the retaining walls. During the wetter months, ground water may be encountered during the construction of Piers 2 and 3.

6. Geologic Hazards

6.1. Site Seismicity

A bedrock acceleration coefficient of 0.30g is recommended for seismic design of the structures on this project in accordance with the 2002 US Geological Survey National Seismic Hazard Map. The recommended acceleration coefficient is based on expected peak bedrock acceleration (PBA) at the project site that has a 90 percent probability of not being exceeded in a 50-year period. We recommend using Type I soil profile response spectrum and a site coefficient (S) of 1.0 for seismic design.

In the past, we have provided only the peak bedrock acceleration (PBA). The PBA should be used for bridge foundation and structural design, as input to develop the response spectra for the structure. The "Peak Ground Acceleration" (PGA) at the ground surface is a function of PBA and the soil profile. Due to soil amplification, we recommend a PGA for compacted fill of 0.35g

(Pier 1), and for Vashon Till, we recommend a PGA of 0.30g (Piers 2 and 3). The PGA should be used for the design of the walls, and was used to assess liquefaction.

6.2. *Liquefaction*

The liquefaction potential of saturated soils is evaluated mainly on soil gradation, density, and the depth of the deposit. The potential for liquefaction is highest for loose, fine to medium grained sands and silty sands. Increasing fines content (i.e., silt and clay) decreases the potential for liquefaction. Clean coarse, grained granular soils (gravels) are also less susceptible to liquefaction due to their high permeability. Typically the potential for liquefaction also decreases with increasing density and depth.

We have evaluated the potential for liquefaction of the project soils based on the SPT data obtained from the field explorations and the percentages of silt and clay. Because the alignment soil either has a high fine content or a medium dense to very dense blow counts, it is our opinion that the potential for liquefaction is low where the bridge and retaining walls are planned. In addition, Washington Division of Geology and Earth Resources Geologic Map GM-47, *Geologic Folio of Olympia-Lacey-Tumwater Urban Area, Washington: Liquefaction Susceptibility Map* by Stephen P. Palmer, Timothy J. Walsh, and Wendy J. Gerstel, January 1999, has identified low to very low susceptibility at the project site.

7. Geotechnical Recommendations

7.1. *Alignment Cut/Fill Recommendations*

It is our understanding that the bridge approach fills will be supported by retaining walls at the end piers. The approach fills should be constructed of gravel borrow and compacted is in accordance with the Standard Specifications. Elsewhere, the cut and fill slopes constructed at 1.75:1 should be stable provided that the fill slopes are constructed of select borrow or gravel borrow, and hillside terraces, as required under Section 2-03.3(14) in the Standard Specifications, are used. Common Borrow may be used on slopes 2:1 or flatter, provided the soil is compacted using Method B and the moisture content does not exceed 3 percent of optimum. However, Common Borrow is not an all weather material and the placement of Common Borrow will likely be limited to only the driest summer months.

Settlement of all fills should occur as the fill is placed. Settlement will be 1-inch or less, and post-construction settlement should be negligible. Slopes at 1.75H:1V may be prone to erosion. We recommend contacting the HQ Landscape Architect regarding appropriate vegetation to be used.

7.2. *Pedestrian Bridge Foundation Recommendations*

The new bridge will be a 241 ft long structure that will be approximately 14.0 ft wide. New approaches will be along the old Chehalis Western Railroad grade. Additional fill will be added behind the end piers to match the new grade. Curtain walls will be constructed on either side of the end pier to retain the new fill. We have evaluated the following foundation options: spread footings and drilled shafts. Spread footings are feasible at all pier locations. An existing pier foundation is known to exist in the vicinity of Pier 2, as shown in Figure A-2. From the as built drawings, the foundation measures 33 long by 12 ft wide by 2 ft deep. The bottom of the pier footing is approximately 9 ft below the ground surface based on test hole H-3-05. Due to the confined workspace at Pier 2 and the presence of the existing footing, a drilled shaft option is likely to have less traffic disruption. However, if traffic can be diverted, a spread footing foundation at Pier 2 may be feasible.

7.2.1. Spread Footings Recommendations

We recommend that spread footings be used to support the bridge. For our analyses, we assumed that the new foundations to be at minimum embedment as required by the Bridge Design Manual. Pier 1 will be placed in medium dense silty sand with gravel. Pier 2 may be constructed on top of the old bridge footing or be placed on Vashon till. Pier 3 will be placed on dense Vashon till.

Load and Resistance Factor Design (LRFD) methodology is currently used by the Bridge Office for structural design. Capacity charts for the Strength, Service and Extreme limit states are shown on Figure D-1. There will be some differential settlement between Pier 1 and Pier 2. We expect Pier 1 may settle up to 1 inch, and Piers 2 and 3 will have negligible settlement. As foundations are loaded, we expect that up to 1.0 inch of differential settlement may occur between Piers 1 and 2.

We recommend that the following resistance factors be used when evaluating the different limit states for shallow foundations.

Table 1: Resistance Factors

Limit State	Resistance Factor ϕ		
	Shear Resistance to Sliding	Bearing	Passive Pressure Resistance to Sliding
Strength	0.80	0.45	0.50
Service	1.00	1.00	1.00
Extreme	0.90	1.00	0.90

For passive pressure resistance at the foundation toe and active pressure acting on the abutments, the soil properties provided on Table 2 should be used to estimate the forces.

Table 2: Soil Properties

Parameter	Pier 1 (Fill)	Pier 3 (Dense Soil)
Unit Weight, γ	130 pcf	135 pcf
Soil Friction Angle, ϕ'_f	36°	38°
Active Earth Pressure Coefficient, K_a	0.26	0.24
At rest Earth Pressure Coefficient, K_o	0.41	0.38
Passive Earth Pressure Coefficient, K_p (Flat Ground – Coulomb's Method)	3.8	4.2
Seismic Earth Pressure Coefficient, K_{ae}	0.35	0.32
Coefficient of Sliding (tan ϕ_f)	0.73	0.78

7.2.2. Soil Springs for Spread Footings

We recommend that equivalent spring constant for the spread footing foundation be determined by the method outlined in section 7.2.4 of the FHWA Report No. FHWA-IP-87-6 entitled: Seismic Design And Retrofit For Highway Bridges. The shear modulus and Poisson's ration of the foundation soil must be estimated to calculate the equivalent spring constant using this method.

Based on the result of our analysis, we have developed a range of shear modulus values for the soil unit under these spread-footing foundations. Our recommended soil parameters for spring constants are provided in the following table:

Table 3: Soil Shear Modulus

Pier Location	Shear Modulus *	Poisson's Ratio, μ
Pier 1	550 to 1660 ksf	0.3
Piers 2 and 3	880 to 2640 ksf	0.3

* Shear modulus is for strain magnitudes expected for strong motion earthquakes between 0.2 to 0.02 percent strain, respectively.

7.2.3. Drilled Shaft Recommendation (Pier 2)

In order to minimize the amount of excavation at Pier 2, we are providing a drilled shaft option. It is our understanding that Bridge is considering drilling two small shafts through an existing concrete footing. The new shaft cap will be placed on top of the existing footing. It is expected that this option will require less excavation, and will be easier to construct under traffic in a constricted space.

In Appendix D, we have provided Drilled Shaft Capacity Charts for shaft diameters that range from 2.5 ft to 4 ft, Figures D-2 through D-5 respectively. At a given depth on the figures, the factored resistance (Q') can be determined by adding the ultimate skin friction (Q_s) multiplied by its resistance factor (ϕ_s), and the ultimate end bearing (Q_b) multiplied by its resistance factor (ϕ_b) as shown in the following equation:

$$Q' = Q_s \times \phi_s + Q_b \times \phi_b$$

For the service limit state, the settlement of the shaft foundations will be less than 1-inch. Settlement will occur as the loads are applied. Post-constructed settlement should be negligible.

7.2.3.1. Resistance Factors for Drilled Shaft Design

We recommend that the following resistance factors be used when evaluating the different limit states.

Table 4: Resistance Factors

Limit State	Resistance Factor ϕ		
	Skin Friction, Q_s	**End Bearing, Q_b**	**Uplift**
Strength	0.55	0.50	0.45
Service	1.00	1.00	N/A
Extreme	1.00	1.00	0.80

7.2.3.2. Lateral Load Analysis

For lateral analysis of foundation elements in a group, reduction factors should be used if P-y methods of analysis are used. The values of P should be multiplied by the values, P_m, in Table 5 to modify the P-y curves used in the analysis. The multipliers, P_m, in Table 5 are a function of the center-to-center spacing expressed in multiples of the foundation element diameter (D) as measured along the direction of loading within the group. The values of P_m in Table 5 were developed for vertical elements only. Note that P_m is not applicable if strain wedge theory is used.

The P-y curve parameters are provided in Figure D-1 in Appendix D.

Table 5: Load Modifiers, P_m, for Multiple Row Shading (averaged from **Hannigan, et al., 1997**).

Center-to-Center spacing in the direction of loading	**Load Modifiers, P_m**		
	Row 1	**Row 2**	**Row 3 and higher**
2.5D	0.65	0.40	0.25
3D	0.70	0.50	0.35
4D	0.85	0.70	0.55
5D	1.00	0.85	0.70

Loading direction and spacing are as defined in Figure 1. Note that if the loading direction for a single row is perpendicular to the row (bottom right detail in the figure), a group reduction factor of less than 1.0 should only be used if the spacing is 5D or less, as shown in the detail.

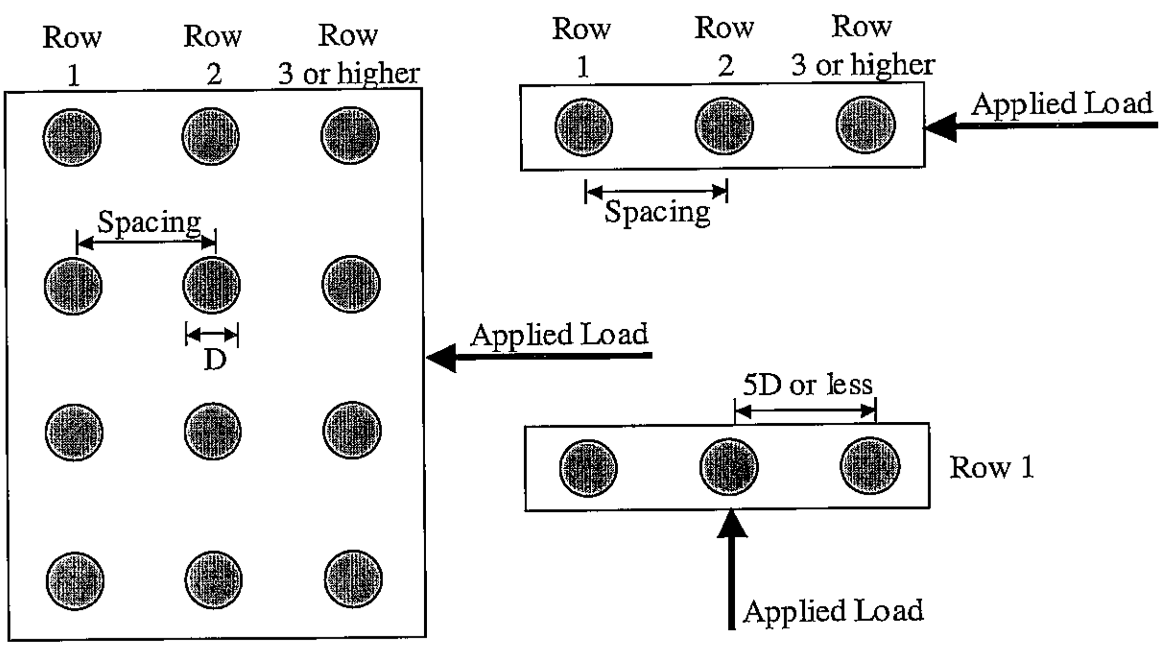


Figure 1: Definition of loading direction and spacing for group effects.

Hannigan, P.J., G.G. Goble, G. Thendean, G.E. Likins and F. Rausche, 1997. "Design and Construction of Driven Pile Foundations" - Vol. I and II, Federal Highway Administration Report No. FHWA-HI-97-013, Federal Highway Administration, Washington, D.C., 822 pp.

7.2.3.3. Potential for Downdrag

Since the shafts will be constructed in glacial till, the potential for downdrag is negligible at this site.

7.3. Retaining Wall Recommendations

7.3.1. General Wall Recommendations

We are providing geotechnical design recommendations for six wall locations, Walls A, B, C, and D at Piers 1 and 2, RW1 and RW2 Walls. Walls A, B, C, and D will be attached to the end piers, and are needed to retain the new approach fills. It is our understanding that these walls will be removed at some future date for expansion of the pedestrian structure. Therefore, the Project Office and Bridge and Structures are considering constructing Standard Plan Permanent Geosynthic Walls or Welded Wire MSE Walls for Walls A, B, C, and D. Standard Plan Reinforced Concrete Walls and other Structural Earth Wall types are feasible at these wall locations. the RW1 Wall will retain new fill next to the USFW building. the RW2 Wall will be a cut wall adjacent to a sidewalk along Lindsley Lane. Recommendations for each wall are discussed below.

7.3.2. Walls A, B, C, and D

The walls will have a maximum exposed height of 22 ft. A Standard Plan Permanent Geosynthic Wall Type 1 is the preferred wall type with a cast-in-place vertical face.

We expect to encounter medium dense to very dense silty sand with gravel at the bearing elevations. The allowable bearing capacity is 8 ksf. Settlement should occur as the wall is constructed with settlements being 1.0 inch or less. Post-construction settlement should be negligible.

7.3.3. RW 1 Wall

RW 1 Wall will be constructed in new fill on the north end of the project between Stations CWT 17+18 and CWT 17+90, Left. The wall length will be approximately 95 ft, and the wall will have a maximum exposed height of 8 ft with a 1.75:1 back slope. Standard Plan Reinforced Concrete Wall, a Standard Plan Permanent Geosynthic Wall Type 3, and other Structural Earth Wall types are feasible at the wall location.

We expect to encounter dense to very dense poorly-graded sand with silty to well-graded gravel with silt and sand at this location. The walls can be designed for an allowable bearing of 8 ksf. Settlement should occur as the fill is placed with post-construction settlement being negligible.

For Structural Earth Walls (SE Walls), AASHTO Bridge Design Specifications require a minimum reinforcement length of 8.0 ft, regardless of wall height. This limitation is primarily due to the size limitations of conventional spreading and compaction equipment. If a pre-approved SE wall is used, the plans will need to require an 8 ft minimum reinforcing length. Geosynthetic walls and SE walls should be founded as discussed in Section 7.3.5. Concrete walls should be founded at an elevation that will provide 2 ft of cover over the top of the foundation.

7.3.4. RW 2 Wall

RW 2 Wall will be a cut wall adjacent to a sidewalk and an adjacent private road that intersects Martin Way in Lacey between Stations CWT 21+60 and CWT 21+71, 6 ft Right. The wall will have an approximate length of 111 ft and a maximum exposed height of 3 ft. It is our understanding that a Modular Block Wall is the preferred wall type. This wall type will match the existing walls in the vicinity of the new wall.

We expect to encounter dense to very dense poorly-graded sand with silty to well-graded gravel with silt and sand at this location. Settlement should occur as the fill is place with post-construction settlement being negligible. Settlement will be 1.0 inch or less.

We have provided a typical design for the Modular Block Wall in the attached Figure D-8. This wall type is a special design that requires a special provision. Attached in Appendix E, is a special provision that should be included in the contract documents.

7.3.5. Wall Design Requirements

The following items should be considered in preparation of contract documents:

1. All walls should be placed on a level foundation in the direction perpendicular to the wall face.
2. Leveling pads and bottoms of SE walls should be located above the water table, which will require using a minimum embedment of 2.0 ft below the final ground surface or 10% of the total wall height, which ever is greater.
3. SE walls should have a wall face batter no steeper than 48V:1H.
4. The base width of the SE walls should not be less than 70 percent of the wall height or 8 feet which ever is greater to insure overall stability. Figure D-7 provides the minimum wall reinforcing length for the walls. Greater wall base widths may be needed to provide adequate overturning, sliding, and internal stability for the walls.
5. Backfill within the reinforced wall prism of the SE and Geosynthetic walls should consist of Gravel Borrow.
6. Properly compacted (Method B) Gravel Borrow or Select Borrow should be used behind the reinforced wall prism depending on weather conditions during construction. Common Borrow is not recommended.

Detailed wall plans and design for the propriety wall options will not be developed until after the contract is awarded. Therefore, the Project Office should prepare wall plan and profile should be prepared for each wall showing the following:

1. A profile of neat-line top and bottom of wall as well as final ground line in front of and final ground line at the back of wall facing at the top of wall.
2. The backfill slope above the wall should be shown in the Plans.

3. A typical cross-section.
4. Generic details for the desired appurtenances, drainage requirements, guardrail post, and/or traffic barrier, which need to be included in the contract PS&E for proprietary walls. Locations of potential conflicts with the soil reinforcement must be shown.
5. A geotextile wrapped under-drain should be provided at the base of the wall behind the reinforced zone. This should be shown in the plans. Figure D-4 shows a typical example.
6. The drainage pipe needs to daylight through the MSE wall or at a sag (low) point or at a maximum 300 ft interval along the wall face.

Ideally the catch basins, grate inlets, and signal foundations should be located outside the reinforced backfill zone of the walls to avoid interference with the soil reinforcement. However, in some cases it may not be possible to do this. In those cases, where conflict with the reinforcement cannot be avoided, the location(s) and dimensions of the reinforcement obstruction(s) relative to the wall must be clearly indicated on the retaining wall plans. The Project Office should contact the Bridge and Structures Office to determine the limits on the size and location of the obstructions for which pre-approved wall details and designs are available, and regarding what generic details to provide in the plans.

Once the detailed wall plans and designs are available as shop drawings after the contract is awarded, the Bridge and Structures Office will need to review and approve the wall shop drawings and calculations.

If a Structural Earth Wall is selected, specific design information needs to be included as part of the Structural Earth Wall GSP. The following design information should be inserted in the GSP for the walls:

Table 6: GSP Fill-ins

Soil Properties	**Wall Backfill**	**Retained Soil**	**Foundation Soil**
Unit Weight (pcf)	130	125	125
Friction Angle (degrees)	36°	34°	36°
Cohesion (psf)	0	0	0
		AASHTO Load Group I	AASHTO Load Group VII
Allowable Bearing Capacity		8 ksf	12 ksf
Peak Ground Acceleration Coefficient (g)		0	0.30

If the permanent geosynthetic wall is selected, we recommend a Standard Plan D-3 Type 1. The geosynthetic wall should be considered a Class 1 structure. We recommend using the current amended Standard Specifications, Sections 6-13, 6-14, and 6-18, and GSPs for construction of the Structural Earth Walls and/or the Permanent Geosynthetic Walls.

The GSPs for construction of both walls are included as attachments in Appendix F. This information is also available on the WSDOT web site under http://www.wsdot.wa.gov/eesc/design/projectdev/gsp. The Geosynetic Wall information is found under 14.ap6, 14.gb6, 1402.gb6, 14021.gb6, and 140201.fb6. The Shotcrete Facing information is found under 18.ap6, 18.gb6, 1802.gb6, and 180201.gb6.

8. Construction Considerations

8.1. Construction Considerations

There are several generalized construction considerations that will require attention during design and construction of this project. In order to maintain traffic, the construction will be done in multiple phases.

The generalized construction considerations are as follows:

1. An existing abandon foundation will be encountered at Pier 2 that measures 33 long by 12 ft wide by 2 ft deep. Steel reinforcing bars were used in construction of the footing. Several construction options have been discussed: remove the old footing, construct the new footing on top of the old footing, or drill small diameter shafts through the old footing. If a spread footing is placed on top of the existing pier, additional excavation may be required to provide full coverage beneath the new footing. We recommend that the excavated cavity be backfilled with concrete Class 4000P. If controlled density fill is used, a weak point between the old concrete and the CDF may develop. If shafts are drilled through the old footing, the contractor should expect hard drilling when they encounter steel reinforcing, especially near the bottom of the existing foundation.

2. Based on the conditions observed during site explorations, we anticipate cobbles and boulders will be encountered below the existing concrete foundation during shaft excavations at Pier 2. Difficult drilling conditions should be expected in the dense and very dense soils.

3. If shoring is required to construct the Pier 2 foundation, hard driving conditions may be encountered. Vashon Till was encountered at a shallow depth at this site. Driving sheet piling or H-piles in well-graded gravel with sand to a depth equal to the design height of the exposed face will not be feasible. During our field investigation, we did not encounter any cobbles and boulders in our test drilling at this location. However, cobbles and boulders may be present in the till soil units based on the geologic history of these soil units. The presence of cobbles and boulders could lead to hard driving conditions. The cobbles and boulders could also affect alignment of the sheets. Drilled methods may be required, and a special shoring plan may need to be developed.

4. If drilled shafts are used, shaft casing will likely not be required. The old concrete foundation and the very dense Vashon Till will in general support the sidewalls of the shaft. However, there may be localized zones of cohesionless soil that could cave. Ground water may be encountered. Therefore, the contractor should expect that wet shaft construction methods will be needed. A sleeve may be required between the old foundation and the shaft. This sleeve will provide a separation barrier to prevent a hard, stiff point near the top of the shaft column.

5. During the excavation for the wall foundations and Bridge Pier 3, ground water seepage may be encountered near the bottom of the excavation. We expect dewatering of the foundation may be required to pour the foundation in the dry.
6. Compaction of the backfill below the water table will be difficult. We recommend using shot rock or quarry spalls for backfill below the water table. The top of the quarry spalls should be choked with Shoulder Ballast or Gravel Borrow before placing the remainder of the fill. The quarry spalls should provide an adequate base so that compaction of the fill can be achieved.

9. Closure

The future performance and integrity of the structure and the geotechnical elements of this project depend largely on proper PS&E preparation and diligent construction procedures. Therefore, we recommend that the E&EP Geotechnical Division (GD) provide the following post-report services:

- The GD should prepare the Summary of Geotechnical Conditions to be included in the PS&E as an appendix. The summary should be prepared as part of the PS&E review process.
- The GD should review all construction plans and specifications to verify that the design criteria presented in this report have been interpreted correctly and properly integrated into the design.
- The GD should attend pre-construction conferences with the Construction Project Engineer and the Contractor to discuss important geotechnical construction issues.
- The GD or the Region Materials Engineer should observe all exposed subgrades for spread footings after completion of stripping and excavation to contract elevations. The GD or the Region Materials Engineer should confirm that suitable soil conditions have been reached and determine appropriate subgrade compaction methods.

10. Intended Report Use and Limitations

This report has been prepared to assist the Washington State Department of Transportation in the engineering design and construction of the subject project. It should not be used, in part or in whole for other purposes without contacting the E&EP Geotechnical Division for a review of applicability of such reuse. This report should be made available to prospective contractors for their information or factual data only and not as a warranty of ground conditions.

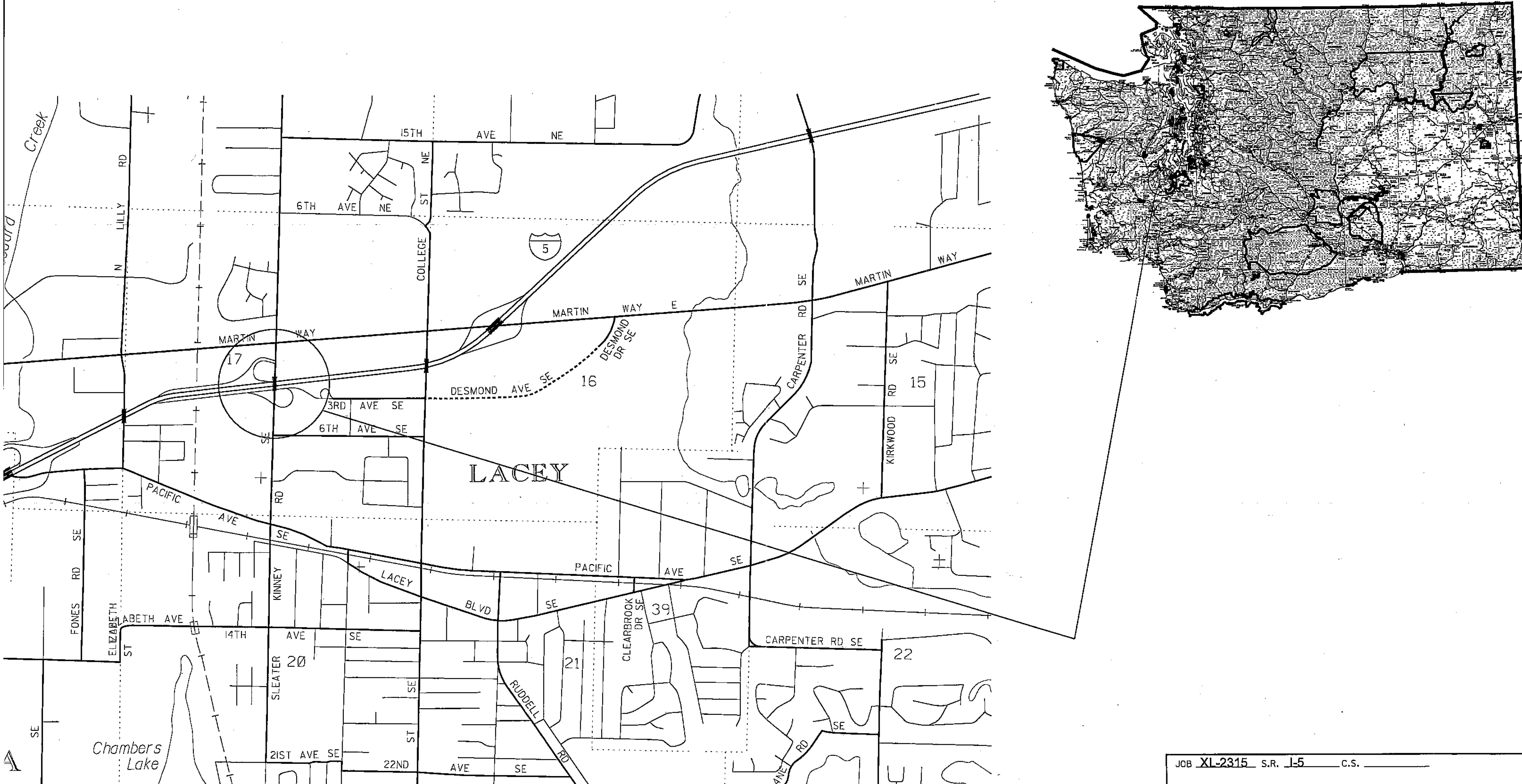
The conclusions and recommendations contained in this report are based on the Geotechnical Division's understanding of the project at the time that the report was written on site conditions that existed at the time of the field exploration. If significant changes to the nature, configuration, or scope of the project occur during the design process, the Geotechnical Division should be consulted to determine the impact of such changes on the recommendations and conclusions presented in this report.

Site exploration and testing describes subsurface conditions only at the sites of subsurface exploration and at intervals where samples are collected. These data are interpreted by members of the Geotechnical Division who render an opinion regarding the general subsurface conditions. The distribution, continuity, thickness, and characteristic of identified (and unidentified) subsurface materials may vary considerably from that indicated by the subsurface data. While nothing can be done to prevent such variability, the Geotechnical Division is prepared to work with the Design Team to reduce the impacts of variability on the project design, construction, and performance. Periodic geotechnical observation during construction may be beneficial in this respect. This ongoing involvement of the Geotechnical Division throughout the design and project development process will also help to avoid shortcomings of project design or contract documents.

The conclusions and recommendations presented in this report assume that surface and subsurface conditions, as observed during field exploration activities, are representative of the site conditions throughout the project area. Accordingly, the Geotechnical Division and/or the Region Materials Engineer should be involved in the construction of the project in order to make appropriate observations and recommendations for alteration in design as appropriate.

APPENDIX - A

Figure A-1	Vicinity Map
Figure A-2	Test Hole Locations
Figure A-3	Soil Profile



JOB XL-2315 S.R. I-5 C.S.

Chehalis Western Trails Pedestrian Bridge

WASHINGTON STATE TRANSPORTATION COMMISSION DEPARTMENT OF TRANSPORTATION MATERIALS BRANCH T.E. BAKER MATERIALS ENGINEER	DATE 6/2005 SCALE N.T.S. VERT. HORIZ. SHEET ___ OF ___ DRAWN BY WM

Figure A-1: Vicinity Map

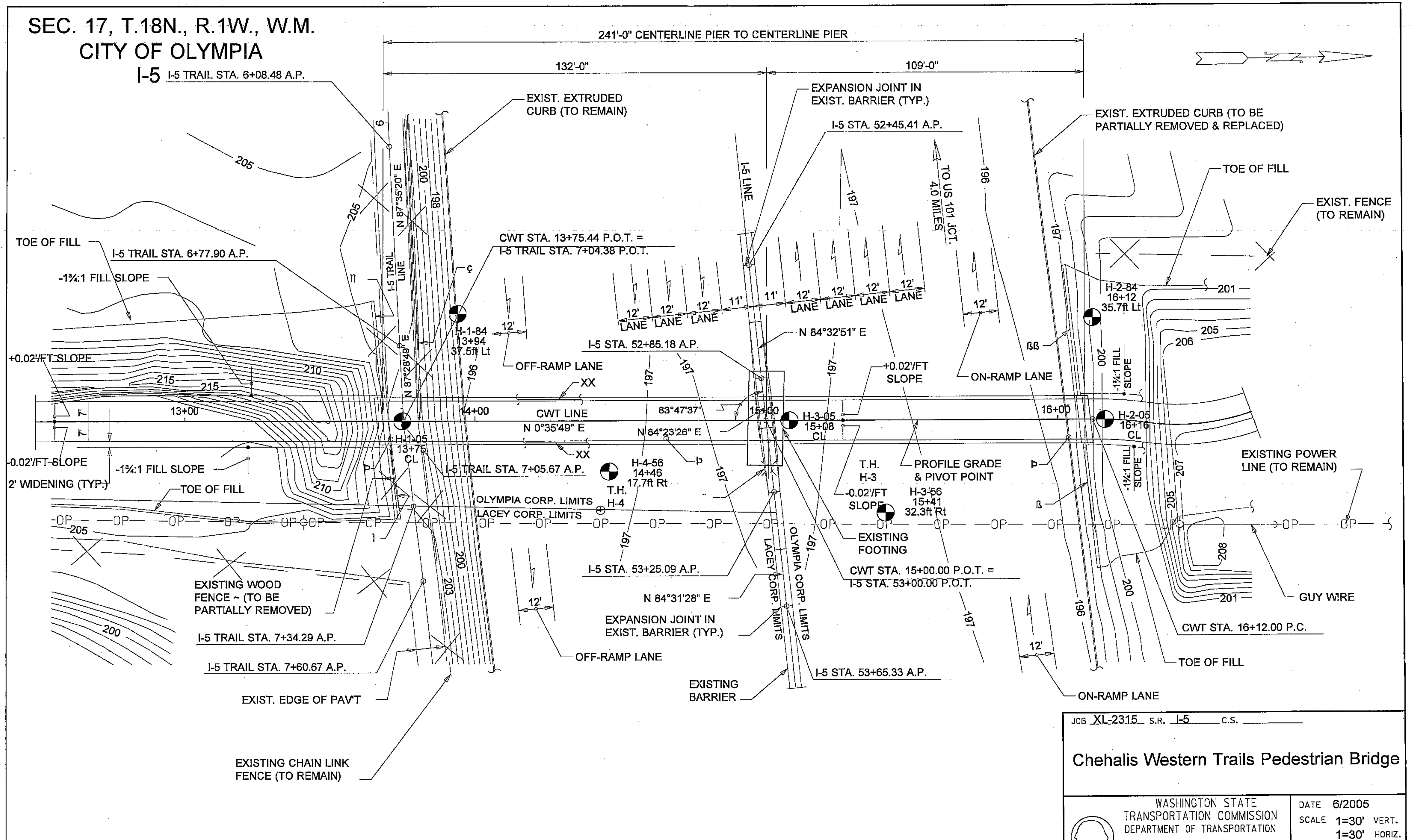


Figure A-2: Test Hole Locations

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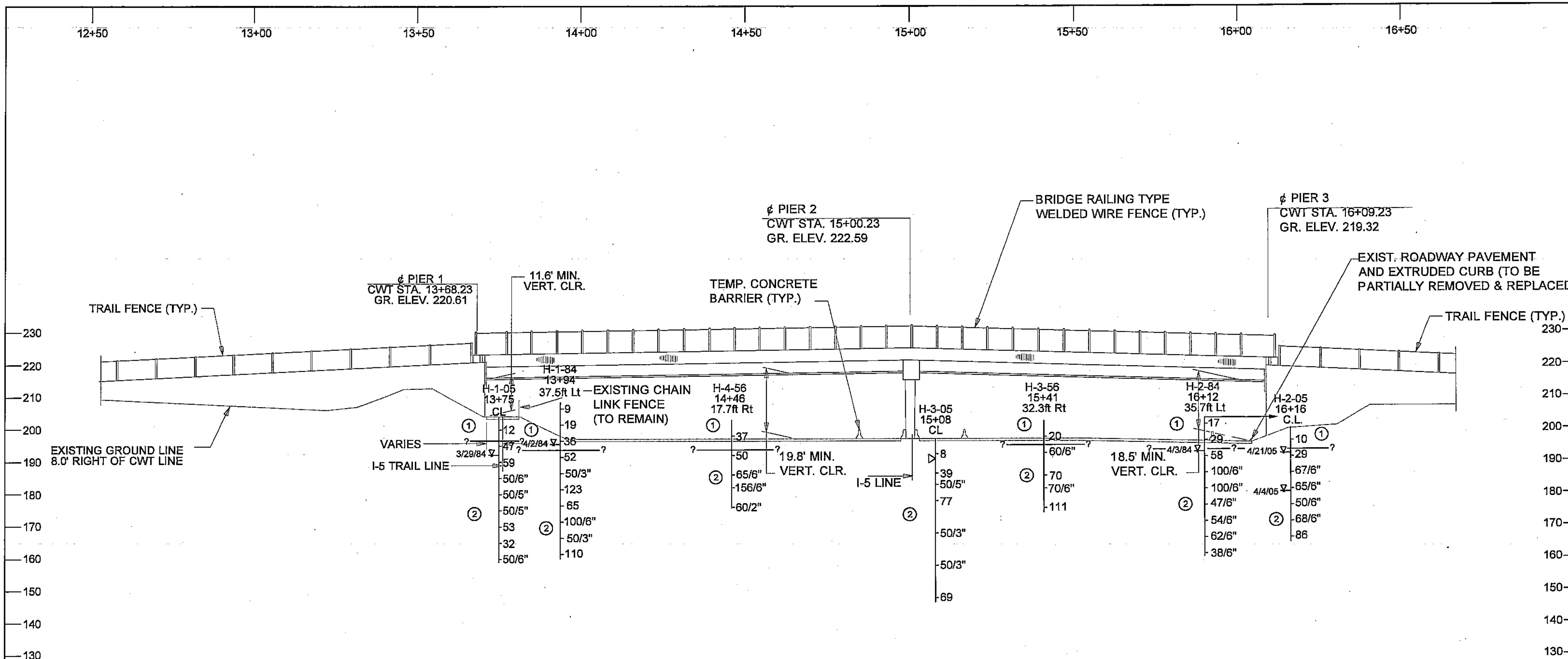


UNIT 1: Loose to dense, silty SAND with gravel

UNIT 2: Dense to very dense, poorly graded GRAVEL with sand and silt to well graded GRAVEL with sand, cobbles, and boulders.

TEST HOLE LEGEND

- H-1-98 — TEST HOLE NUMBER
- 110+55 — TEST HOLE STATION
- 26 ft.Rt. — TEST HOLE OFFSET
- 23 — STANDARD PENETROMETER TEST (BLOWS PER FOOT)
- UNDISTURBED SAMPLE
- W.L. 8-6-86 — WATER LEVEL & DATE
- ? — ? INDICATES SOIL/ROCK STRATA BETWEEN TEST HOLES MAY NOT BE CONTINUOUS
- INDICATES INTACT ROCK
- 86% — INDICATES CORE SAMPLE TAKEN
- ROCK QUALITY DESIGNATION

DATUM
NAVD 1988

JOB XL-2315 S.R. I-5 C.S. 3401

Chehalis Western Trail Pedestrian Bridge

WASHINGTON STATE
TRANSPORTATION COMMISSION
DEPARTMENT OF TRANSPORTATION
MATERIALS BRANCH
T. E. BAKER MATERIALS ENGINEER

DATE 6/2005
SCALE 1=30' VERT.
1=30' HORIZ.
SHEET ____ OF ____
DRAWN BY WM

Figure A-3: Soil Profile

...\WESTERN TRAIL BSD.dgn 08/11/2005 09:17:41 AM

APPENDIX - B

Logs of Test Borings

Washington State
Department of Transportation

Test Boring Legend

Sampler Symbols

Standard Penetration Test
Oversized Penetration Test (Dames & Moore, California)
Shelby Tube
Piston Sample
Washington Undisturbed
Vane Shear Test
Core
Becker Hammer
Bag Sample

Well Symbols

Cement Surface Seal
Piezometer Pipe in Granular Bentonite Seal
Piezometer Pipe in Sand
Well Screen in Sand
Granular Bentonite Bottom Seal
Inclinometer Casing in Concrete Bentonite Grout

Laboratory Testing Codes

UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
UC	Unconfined Compression Test
DS	Direct Shear Test
CN	Consolidation Test
GS	Grain Size Distribution
MC	Moisture Content
SG	Specific Gravity
OR	Organic Content
DN	Density
AL	Atterberg Limits
PT	Point Load Compressive Test
SL	Slake Test
DG	Degradation
LA	LA Abrasion
HT	Hydrometer Test

Soil Density Modifiers

Gravel, Sand & Non-plastic Silt		Elastic Silts and Clay	
SPT Blows/ft	Density	SPT Blows/ft	Consistency
0-4	Very Loose	0-1	Very Soft
5-10	Loose	2-4	Soft
11-24	Medium Dense	5-8	Medium Stiff
25-50	Dense	9-15	Stiff
>50	Very Dense	16-30	Very Stiff
		31-60	Hard
		>60	Very Hard

Angularity of Gravel & Cobbles

Angular	Coarse particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Coarse grained particles are similar to angular but have rounded edges.
Subrounded	Coarse grained particles have nearly plane sides but have well rounded corners and edges.
Rounded	Coarse grained particles have smoothly curved sides and no edges.

Soil Moisture Modifiers

Dry	Absence of moisture; dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible free water

Soil Structure

Stratified	Alternating layers of varying material or color at least 6mm thick; note thickness and inclination.
Laminated	Alternating layers of varying material or color less than 6mm thick; note thickness and inclination.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, somtimes striated.
Blocky	Cohesive soil that can be broken down into smaller angular lumps which resist further breakdown.
Disrupted	Soil structure is broken and mixed. Infers that material has moved substantially - landslide debris.
Homogeneous	Same color and appearance throughout.

HCL Reaction

No HCL Reaction	No visible reaction.
Weak HCL Reaction	Some reaction with bubbles forming slowly.
Strong HCL Reaction	Violent reaction with bubbles forming imediately.

Degree of Vesicularity of Pyroclastic Rocks

Slightly Vesicular	5 to 10 percent of total
Moderately Vesicular	10 to 25 percent of total
Highly Vesicular	25 to 50 percent of total
Scoriaceous	Greater than 50 percent of total

Washington State
Department of Transportation

Test Boring Legend

Grain Size

Fine Grained	< 1mm	Few crystal boundaries/grains are distinguishable in the field or with hand lens.
Medium Grained	1mm to 5mm	Most crystal boundaries/grains are distinguishable with the aid of a hand lens.
Coarse Grained	> 5mm	Most crystal boundaries/grains are distinguishable with the naked eye.

Weathered State

Term	Description	Grade
Fresh	No visible sign of rock material weathering; perhaps slight discoloration in major discontinuity surfaces.	I
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than its fresh condition.	II
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a continuous framework or as core stones.	III
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as discontinuous framework or as core stone.	IV
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual Soil	All rock material is converted to soil. The mass structure and material fabric is destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Relative Rock Strength

Grade	Description	Field Identification	Uniaxial Compressive Strength approx
R1	Very Weak	Specimen crumbles under sharp blow from point of geological hammer, and can be cut with a pocket knife.	150-3500 psi
R2	Moderately Weak	Shallow cuts or scrapes can be made in a specimen with a pocket knife. Geological hammer point indents deeply with firm blow.	3500-7500 psi
R3	Moderately Strong	Specimen cannot be scraped or cut with a pocket knife, shallow indentation can be made under firm blows from a hammer.	7500-15000 psi
R4	Strong	Specimen breaks with one firm blow from the hammer end of a geological hammer.	15000-350000 psi
R5	Very Strong	Specimen requires many blows of a geological hammer to break intact sample.	Greater than 30000 psi

Discontinuities

Spacing	
Very Widely	Greater than 3 m
Widely	1 m to 3 m
Moderately	0.3 m to 1 m
Closely	50 mm to 300 mm
Very Closely	Less than 50 mm

RQD (%)

$$\frac{100(\text{length of core in pieces} > 100\text{mm})}{\text{Length of core run}}$$

Condition	
Excellent	Very rough surfaces, no separation, hard discontinuity wall
Good	Slightly rough surfaces, separation less than 1 mm, hard discontinuity wall.
Fair	Slightly rough surfaces, separation greater than 1 mm, soft discontinuity wall.
Poor	Slickensided surfaces, or soft gouge less than 5 mm thick, or open discontinuities 1 to 5 mm.
Very Poor	Soft gouge greater than 5 mm thick, or open discontinuities greater than 5 mm.

Fracture Frequency (FF) is the average number of fractures per 300 mm of core.
Does not include mechanical breaks caused by drilling or handling.

Definitions of Field Explorations and Instrumentation

In the test hole either a standard penetration test (SPT) or an undisturbed sample were performed and samples taken at between 5-foot to 10-foot intervals. In fine-grained soils, either a Shelby tube or Washington Undisturbed Sampler was pushed followed by a SPT. The disturbed soil samples were visually identified in the field and then submitted to the Materials Laboratory for a more detailed classification.

The Standard Penetration Test (SPT) is the most widely used method for determining soil conditions in the world. Disturbed soil samples were obtained in general accordance with ASTM D-1586. SPTs are obtained by driving a 2-inch outside diameter split-spoon sampler 18 inches into the soil with a 140-pound weight falling a distance of 30 inches. The number of blows required to achieve each 6 inches of penetration is recorded and the soil's SPT resistance, or N-value, is calculated as the number of blows required to achieve the final 12 inches of penetration. The samples are designated by the preface of D.

A portable-penetrometer test is a dynamic penetration test, which is a derivative of the SPT test. The penetrometer consists of a cone-shaped tip attached to dill rod sections (one inch OD steel pipe), and a hammer section. The hammer section consists of a 35-lb weight free falling 25.5 inches and impacting on the drill rod coupling device. The weight is manually raised and allowed to free fall and impact the dill rods. The number of blows per six inches is counted. The test holes are designated by the preface of PP.

A Washington Undisturbed Sampler consists of a 2.4-inch outside diameter steel tube sampler with brass tube liners that is pushed into fine-grained soils using hydraulic pressure to obtain relatively undisturbed samples. The brass tubes that fit inside the sampler are 4 inches long and have inside diameters of 1.915 inches. The samples are designated by the preface of U.

Shelby tube undisturbed sampler consist of either a 2.0 inch or 3.0 inch outside diameter thin walled steel tube that is pushed into fine-grained soils using hydraulic pressure. The tube is 30 inches long with a wall thickness of approximately 0.067 inches with a beveled end that acts like a cookie cutter. The samples are designated by the preface of S.

Open standpipe piezometers are constructed of 2-inch diameter Schedule 40 PVC pipe with a 2-foot long screened section for a tip. Piezometer tips were surrounded with pea gravel. A seal of either bentonite or Hole Plug was used to cap the pea gravel from the ground surface. In general each piezometer was enclosed at the surface by a steel monument seated is concrete. A water level is measured by a water level indicator device, which consists of a graduated cable with a steel weight at its lower end. The upper end of the cable I connected to a battery operated indicator light and/or buzzer. When the probe is lowered into the standpipe and encounters the water surface, the electrical circuit is complete and the length of cable can be measured.

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card S-22719
HOLE No. H-1-05
Sheet 1 of 3

Job No. XL-2315 SR 5 Elevation 203.9 ft (62.1 m)

Project Chehalis Western Trail Pedestrian Bridge

Driller Robert Shepherd Lic# 2710T

Site Address

Inspector James Fetterly

Start March 29, 2005 Completion March 29, 2005 Well ID#

Equipment CME 55 w/ autohammer

Station 13+75 Offset CL Casing 3.5"

Method Wet Rotary

Northing 10 Easting 15 Latitude

Longitude

County Thurston Subsection NE/SW Section 17 Range 1 WWM Township 18 N

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
5	1 – 2			3 5 7 (12)		D-1	GS MC	SM, M.C. =29% Silty SAND, Stratified with clean poorly graded sand, medium dense, brown, moist, Stratified, HCl reaction not tested, possible fill material Length Recovered 1.5 ft, Length Retained 1.0 ft		
10	3			18 21 26 (47)		D-2		Well graded GRAVEL with silt and sand, subrounded, dense, grey, moist, Homogeneous, HCl reaction not tested, trace FEO stains throughout Length Recovered 1.0 ft, Length Retained 1.0 ft		
								03/29/2005	∇	
15	4 – 5		>>	27 37 22 (59)		D-3	GS MC	GW-GM, M.C. = 9% Well graded GRAVEL with silt and sand, subrounded, very dense, grey, moist, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft		
20	6			14 50/6		D-4		Well graded GRAVEL with sand, subrounded, very dense, grey, moist, Homogeneous, HCl reaction not		

Washington State Department of Transportation

LOG OF TEST BORING

Start Card S-22719

HOLE No. H-1-05

Job No. XL-2315 SR 5 Elevation 203.9 ft (62.1 m)

Sheet 2 of 3

Project Chehalis Western Trail Pedestrian Bridge

Driller Robert Shepherd Lic# 2710T

Depth (ft): 25, 30, 35, 40, 45 — Meters (m): 7, 8, 9, 10, 11, 12, 13

Standard Penetration Blows/ft: 10 20 30 40

SPT Blows/6" (N)	Sample No. (Tube No.)	Lab Tests	Description of Material
(50/6")			tested Length Recovered 1.0 ft, Length Retained 1.0 ft
20 50/5" (50/5")	D-5		Well graded GRAVEL with sand, subrounded, very dense, light grey, wet, Homogeneous, HCl reaction not tested Length Recovered 0.8 ft, Length Retained 0.8 ft
25 50/5" (50/5")	D-6		Well graded GRAVEL with sand, subrounded, very dense, light grey, wet, Homogeneous, HCl reaction not tested Length Recovered 0.6 ft, Length Retained 0.6 ft
7 17 36 (53)	D-7	GS MC	GW, M.C. = 13% Well graded GRAVEL with sand, subrounded, very dense, grey, wet, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft
16 17 15 (32)	D-8		Well graded GRAVEL with sand, subrounded, dense, light brown, wet, Homogeneous, HCl reaction not tested Length Recovered 1.2 ft, Length Retained 1.2 ft
20 50/6	D-9	GS MC	GW, M.C. = 12% Well graded GRAVEL with sand, subrounded, very

LOG OF TEST BORING

Start Card S-22719

HOLE No. H-1-05

Job No. XL-2315 SR 5 Elevation 203.9 ft (62.1 m)

Sheet 3 of 3

Project Chehalis Western Trail Pedestrian Bridge

Driller Robert Shepherd Lic# 2710T

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				(50/6")				dense, grey, wet, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft		
	14							End of test hole boring at 45 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. Bailed hole to 35.2', Fifteen min.recharged to 27.7' and Charged to 11.8' after 1/2 hour.		
	15									
50										
	16									
55										
	17									
	18									
60										
	19									
65										
	20									
	21									
70										

Washington State Department of Transportation

LOG OF TEST BORING

Start Card R-65963

HOLE No. H-2-05

Sheet 1 of 2

Job No. XL-2315 SR 5 Elevation 200.4 ft (61.1 m)

Project Chehalis Western Trail Pedestrian Bridge

Driller Sean Verlo Lic# 2516

Site Address

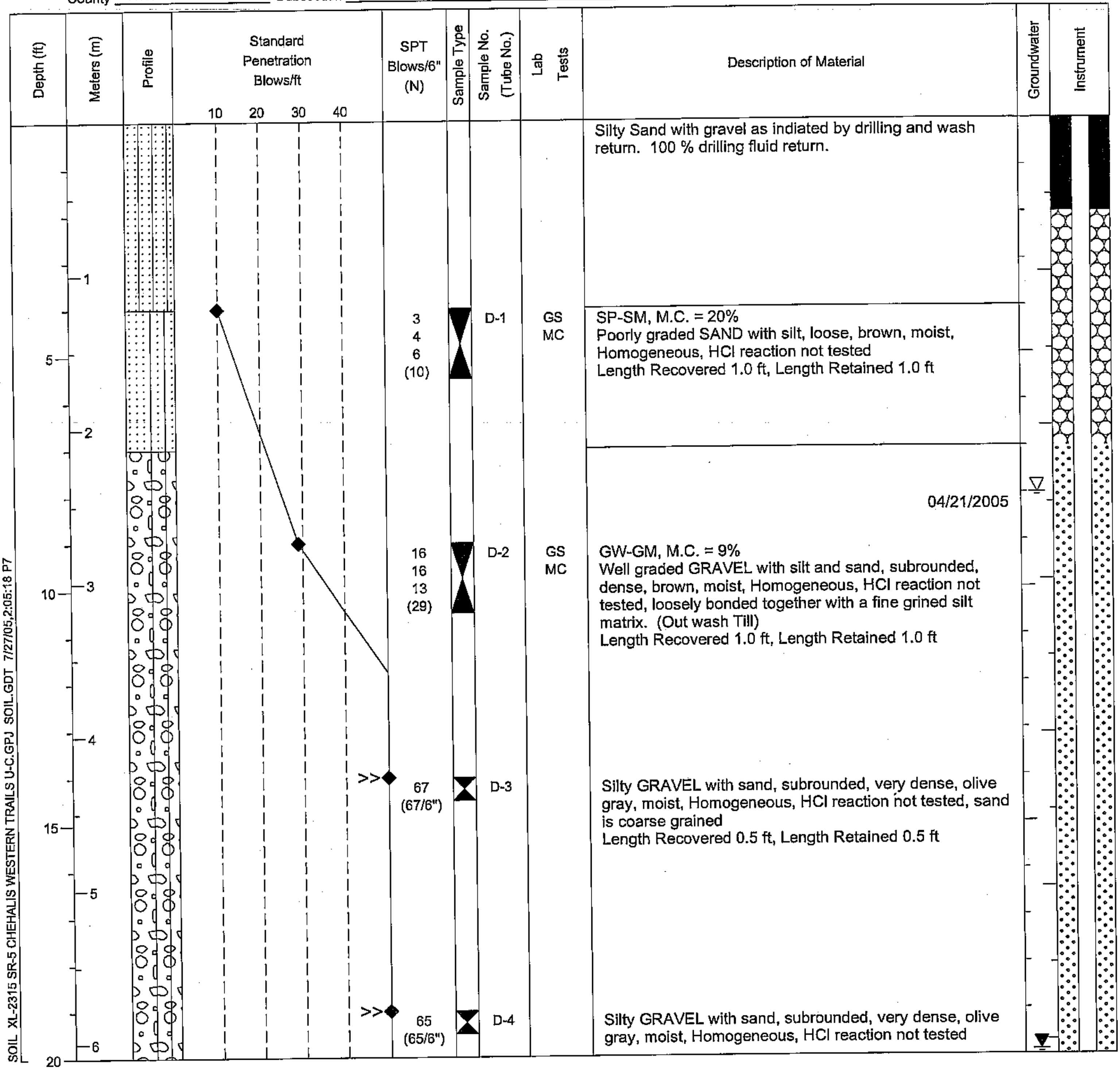
Inspector Cleo Andrews

Start March 30, 2005 Completion March 31, 2005 Well ID# AHN-933 Equipment CME 45 w/ autohammer

Station 16+16 Offset C.L. Casing (HWT4"x22')(HQ3"x40') Method Wet Rotary

Northing 5 Easting 10 Latitude Longitude

County Thurston Subsection NE 1/4 of the SW 1/4 Section 17 Range 1 WWM Township 18 N



Depth (ft)	SPT Blows/6" (N)	Sample No. (Tube No.)	Lab Tests	Description of Material
0				Silty Sand with gravel as indiated by drilling and wash return. 100 % drilling fluid return.
~4	3 4 6 (10)	D-1	GS MC	SP-SM, M.C. = 20% Poorly graded SAND with silt, loose, brown, moist, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft
~7				04/21/2005
~9	16 16 13 (29)	D-2	GS MC	GW-GM, M.C. = 9% Well graded GRAVEL with silt and sand, subrounded, dense, brown, moist, Homogeneous, HCl reaction not tested, loosely bonded together with a fine grined silt matrix. (Out wash Till) Length Recovered 1.0 ft, Length Retained 1.0 ft
~14	67 (67/6")	D-3		Silty GRAVEL with sand, subrounded, very dense, olive gray, moist, Homogeneous, HCl reaction not tested, sand is coarse grained Length Recovered 0.5 ft, Length Retained 0.5 ft
~19	65 (65/6")	D-4		Silty GRAVEL with sand, subrounded, very dense, olive gray, moist, Homogeneous, HCl reaction not tested

SOIL XL-2315 SR-5 CHEHALIS WESTERN TRAILS U-C.GPJ SOIL.GDT 7/27/05,2:05:18 P7

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card R-65963

HOLE No. H-2-05

Job No. XL-2315 SR 5 Elevation 200.4 ft (61.1 m)

Sheet 2 of 2

Project Chehalis Western Trail Pedestrian Bridge

Driller Sean Verlo Lic# 2516

Depth (ft)	Meters (m)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material
						Length Recovered 0.5 ft, Length Retained 0.5 ft 04/04/2005
	7					
25		26 50 (50/6")		D-5		Well graded GRAVEL with sand, subrounded, very dense, olive gray, moist, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft
	8					
30	9	>> 68 (68/6")		D-6		Silty GRAVEL with sand, subrounded, very dense, gray, moist, Homogeneous, HCl reaction not tested, sand is fine to coarse grained Length Recovered 0.5 ft, Length Retained 0.5 ft
	10					
35		>> 41 38 48 (86)		D-7		Silty GRAVEL with sand, subrounded, very dense, gray, wet, Homogeneous, HCl reaction not tested, sand is coarse grained. Length Recovered 1.0 ft, Length Retained 1.0 ft
	11					End of test hole boring at 35.5 ft below ground elevation.
						This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.
40	12					Water table in casing before bailing is 8.5'. Bailed hole to 19.8' after 15 minutes delay water table recharged to 19.9'. Installed Piezo well at 35,0' with 10.0' slotted screen. Water table 4/4/05 is 19.9'.
	13					
45						

Standard Penetration Blows/ft: 10 20 30 40

WATER TABLE

DATE	DEPTH	ELEV.
4/4/05	19.9 ft	180.5 ft
4/21/05	8.0 ft	192.4 ft
6/10/05	9.0 ft	191.4 ft
7/26/05	10.0 ft	189.5 ft

Washington State Department of Transportation

LOG OF TEST BORING

Start Card S-22719

HOLE No. H-3-05

Sheet 1 of 3

Job No. XL-2315 SR 5 Elevation 196.8 ft (60.0 m)

Project Chehalis Western Trail Pedestrian Bridge

Driller Robert Shepherd Lic# 2710T

Site Address

Inspector James Fetterly

Start March 30, 2005 Completion March 30, 2005 Well ID#

Equipment CME 55 w/ autohammer

Station 15+08 Offset CL Casing 3.5"

Method Wet Rotary

Northing 15 Easting 25 Latitude Longitude

County Thurston Subsection NE/SW Section 17 Range 1 WWM Township 18 N

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	1									
5				5 4 4 (8)		D-1		Silty GRAVEL with sand, subrounded, loose, brown, moist, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft		
	2					C-2		Well graded GRAVEL, with concrete, subrounded, dense, grey, moist, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft		
								Concrete fragments		
10	3			17 16 23 (39)		D-3	GS MC	GP-GM, M.C. = 7% Poorly graded GRAVEL with silt and sand, subrounded, dense, light brown, wet, Homogeneous, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft		
	4			37 50/5" (50/5")		D-4		Well graded GRAVEL with sand, subrounded, very dense, grey, wet, Homogeneous, HCl reaction not tested Length Recovered 0.6 ft, Length Retained 0.6 ft		
15	5									
	6		>>	21 44		D-5		Poorly graded GRAVEL with silt and sand, subrounded, very dense, grey, wet, Homogeneous, HCl reaction not		
20										

LOG OF TEST BORING

Washington State Department of Transportation

Start Card S-22719

HOLE No. H-3-05

Sheet 2 of 3

Job No. XL-2315 SR 5 Elevation 196.8 ft (60.0 m)

Project Chehalis Western Trail Pedestrian Bridge

Driller Robert Shepherd Lic# 2710T

Depth (ft)	Meters (m)	SPT Blows/6" (N)	Sample No. (Tube No.)	Description of Material
		33 (77)		tested Length Recovered 1.2 ft, Length Retained 1.2 ft
25	7, 8			
30	9	40 50/3" (50/3")	D-6	Well graded GRAVEL with sand, subrounded, very dense, grey, moist, Homogeneous, HCl reaction not tested Length Recovered 0.7 ft, Length Retained 0.6 ft
35	10, 11			
40	12	37 50/3" (50/3")	D-7	Well graded GRAVEL with sand, subangular, very dense, grey, wet, Homogeneous, HCl reaction not tested Length Recovered 0.5 ft, Length Retained 0.5 ft
45	13			

Standard Penetration Blows/ft: 10 20 30 40

SOIL XL-2315 SR-5 CHEHALIS WESTERN TRAILS U-C.GPJ SOIL.GDT 7/27/05,2:05:19 P7

Washington State Department of Transportation

LOG OF TEST BORING

Start Card S-22719

HOLE No. H-3-05

Sheet 3 of 3

Job No. XL-2315 SR 5 Elevation 196.8 ft (60.0 m)

Project Chehalis Western Trail Pedestrian Bridge

Driller Robert Shepherd Lic# 2710T

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	14									
	15		>>	26 44 25 (69)		D-8		Well graded GRAVEL with sand, subangular, very dense, grey, wet, Stratified, HCl reaction not tested Length Recovered 1.0 ft, Length Retained 1.0 ft		
50								End of test hole boring at 50.5 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
	16									
55	17									
	18									
60	19									
65	20									
	21									
70										

SOIL XL-2315 SR-5 CHEHALIS WESTERN TRAILS U-C.GPJ SOIL.GDT 7/27/05,2:05:19 P7

LOG OF TEST BORING

WASHINGTON S[illegible]TE DEPARTMENT OF TRANSPORTATION

S.H. ____ S.R. 5 SECTION Trosper Rd. I/C to Martin Way I/C Job No. L-6941 (XL-2315)

Hole No. H-1 Sub Section Weyerhaeuser R.R. U-Xing Cont. Sec. 3407

Station W.R.R. 7 + 22 (CTW 13+94) Offset ℄ (37.5' LT) Ground El. 208.4 FT

Type of Boring Augers Casing Augers to 47' W.T. El. 195.4 FT

Inspector ____ Date April 2, 1984 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	9		4 5 4 4 STD PEN 1	Loose, brown, moist, very silty, fine to medium SAND-with a trace of organic material.
5				
	19		5 8 11 14 STD PEN 2	Medium dense, gray and brown, moist, fine gravelly, very silty, fine to coarse SAND.
10				
	36		7 14 22 STD PEN 3	Dense, gray and brown, wet, slightly silty, fine to coarse sandy GRAVEL.
15				
	52		27 29 23 STD PEN 4	Very dense, gray and brown, wet, silty, fine to coarse sandy GRAVEL.
20				

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

DOT FORM 351-003 REVISED 12/79

Copy to ____

Form 351-003-a (H. F. 26,66-A).
Revised 5-67.

Hole No. H-1 Sub Section Weyerhaeuser R.R. U-xing Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	50/3"		51 50/3" STD PEN 5	Very dense, gray and brown, wet, slightly silty, fine to coarse sandy GRAVEL.
25				
				Augers at 27' bailed water down to 17'. Water returned to 13' in 20 minutes.
	123		21 43 80 STD PEN 6	Very dense, gray and brown, wet, slightly silty, fine to coarse sandy GRAVEL-with cobbles.
30				
	65		22 27 38 STD PEN 7	Very dense, gray and brown, wet, fine to coarse sandy GRAVEL-with cobbles.
35				
				Penetrometer driving on boulder.
	100/6"		12 100 STD PEN 8	Very dense, gray and brown, wet, fine to coarse sandy GRAVEL-with cobbles and boulders.
40				
	50/3"		60 50/3" STD PEN 9	Very dense, gray and brown, wet, fine to coarse sandy GRAVEL-with cobbles.
45				

Y Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. H-1 Sub Section Weyerhaeuser R.R. U-Xing Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	110		22 40 70 STD PEN 10	Very dense, gray and brown, wet, slightly silty, fine to coarse sandy GRAVEL-with cobbles.
50				
				Test boring stopped 48'6" below ground elevation.
				This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.

LOG OF TEST BORING

WASHINGTON ST[illegible]TE DEPARTMENT OF TRANSPORTATION

S.H. ____ S.R. I-5 SECTION Trosper Rd. I/C to Martin Way I/C Job No. L-6941 (XL-2315)

Hole No. H-2 Sub Section Weyerhaeuser R.R. U-Xing Cont. Sec. 3407

Station W.R.R. 9 + 42 (CWT 16+12) Offset 3' Lt. ℄ (35.7' LT) Ground El. 203.4 FT

Type of Boring Augers Casing Augers to 42' W.T. El. 192.9 FT

Inspector ____ Date April 3, 1984 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	17		6 8 9 12 STD PEN 1	Medium dense, brown, moist, fine gravelly, silty, fine to coarse SAND.
5				
	29		12 16 13 18 STD PEN 2	Dense, gray and brown, moist, silty, fine to coarse sandy GRAVEL.
10				
	58		14 24 34 STD PEN 3	Very dense, gray and brown, wet, silty, fine to coarse sandy GRAVEL-with cobbles.
15				
	100/6"		100 STD PEN 4	Very dense, gray and brown, wet, slightly silty, fine to coarse sandy GRAVEL-with cobbles.
20				

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

DOT FORM 351-003 REVISED 12/79

Copy to ____

Y Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. H-2 Sub Section Weyerhaeuser R.R. U-Xing Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	100/6"		100 STD PEN 5	Very dense, gray, wet, slightly silty, fine to coarse sandy GRAVEL-with cobbles.
25				
	47/6"		36 47 STD PEN 6	Very dense, gray, wet, gravelly, fine to coarse SAND.
30				
	54/6"		43 54 STD PEN 7	Very dense, gray, wet, gravelly, fine to coarse SAND.
35				
	62/6"		38 62 STD PEN 8	Very dense, gray, wet, gravelly, fine to coarse SAND.
40				
	38/6"		30 38 STD PEN 9	Very dense, gray, wet, fine to coarse sandy GRAVEL
45				Test boring stopped 43' below ground elevation.

WY Form 351-003-a (H. F. 26-66-A).
Revised 5-67.

Hole No. H-2 Sub Section Weyerhaeuser R.R. U-Xing Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				This is a summary Log of Test Boring.
				Soil/Rock descriptions are derived
				from visual field identifications
				and laboratory test data.

LOG OF TEST BORING

WASHINGTON S[illegible]TE DEPARTMENT OF TRANSPORTATION

S.H. ______ S.R. 5 SECTION Trosper Rd. I/C to Martin Way I/C Job No. L-6941 (XL-2315)

Hole No. 3 Sub Section Weyerhaeser R.R. Undercrossing Cont. Sec. 3407

Station W.R.R. 8 + 72.50 (CWT 15+41) Offset 54' Rt. (32.3' RT) Ground El. 202.6 FT

Type of Boring unknown Casing ______ W.T. El. Not determined

Inspector ______ Date Circa 1956 Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				Clayey TOPSOIL.
5	20			
				Brown, medium SAND.
				SAND mixed with gravel.
10	60/6"			
15				
	70			
20				

DOT FORM 351-003 REVISED 12/79

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

Copy to ______

Y Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. 3 Sub Section Weyerhaeser R.R. Undercrossing Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	70/6"			
25				
	111			
30				

LOG OF TEST BORING

WASHINGTON S[illegible]E DEPARTMENT OF TRANSPORTATION

S.H. ____ S.R. 5 SECTION Trosper Rd. I/C to Martin Way I/C

Job No. L-6941 (XL-2315

Hole No. 4 Sub Section Weyerhaeuser R.R. Undercrossing

Cont. Sec. 3407

Station W.R.R. 7 + 73.50 (CWT 14+46) Offset 66.5' Rt. (17.7' RT)

Ground El. 202.9 FT

Type of Boring unknown Casing ____

W.T. El. Not determined

Inspector ____ Date Circa 1956

Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				Gravelly SAND.
5	37			
10				
	50			
				Clayey GRAVEL.
15				
	65/6"			
20				

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Administrator

DOT FORM 351-003 REVISED 12/79

Copy to ____

WY Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. 4 Sub Section Weyerhaeuser R.R. Undercrossing Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	156/6"			
25				
	60/2"			
30				

APPENDIX - C

Laboratory Test Data

Job No. **XL-2315** Date **July 28, 2005**

Hole No. **H-1-05** Sheet **1** of **1**

Project **Chehalis Western Trail Pedestrian Bridge**

Laboratory Summary

Washington State Department of Transportation

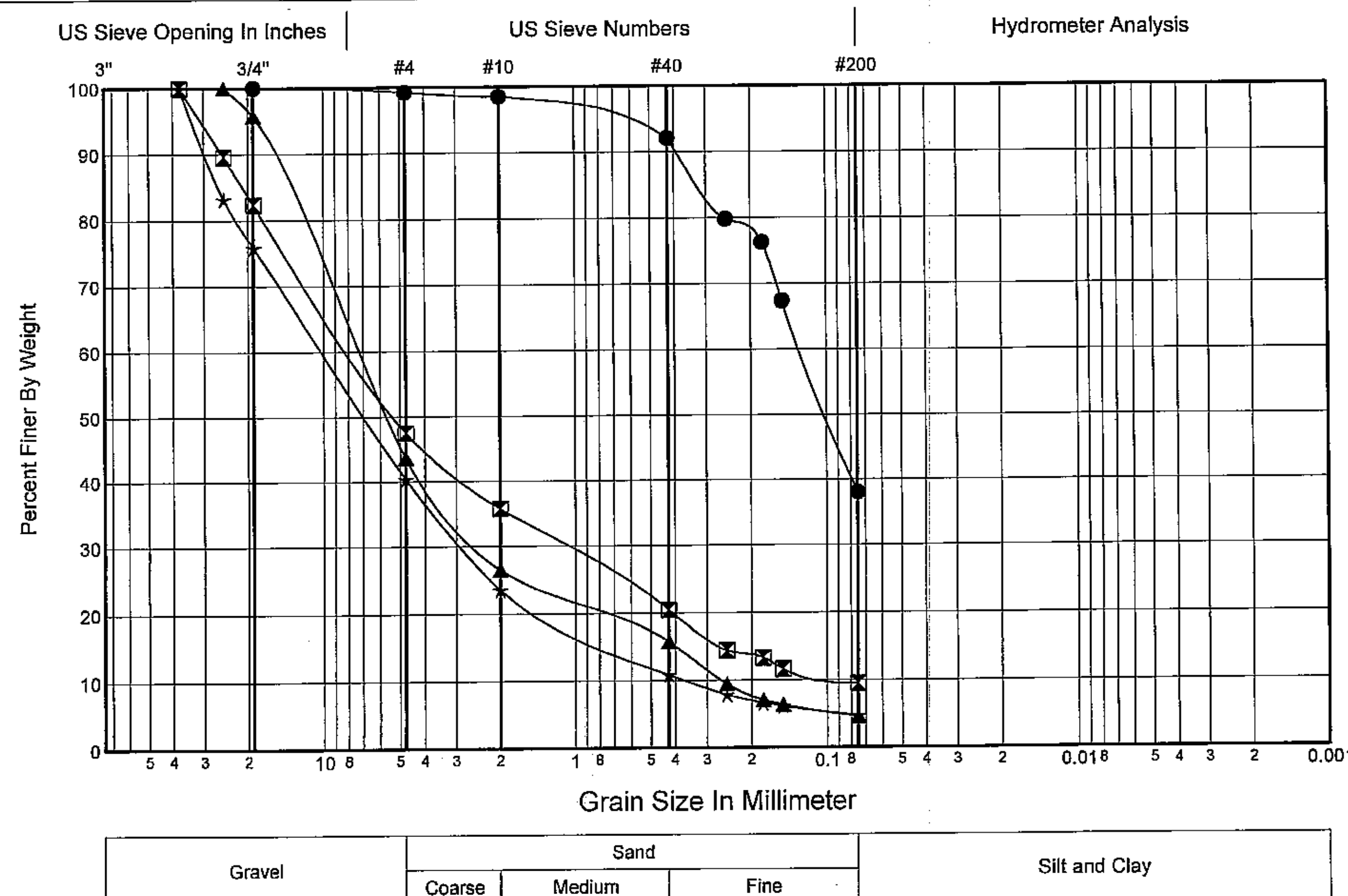
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	4.0	1.22	D-1	SM	See Boring Log	SILTY SAND	29			
⊠	14.0	4.27	D-3	GW-GM	See Boring Log	WELL-GRADED GRAVEL with SILT and SAND	9			
▲	34.0	10.36	D-7	GW	See Boring Log	WELL-GRADED GRAVEL with SAND	13			
★	44.0	13.41	D-9	GW	See Boring Log	WELL-GRADED GRAVEL with SAND	12			

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	0.8	61.2	38.0		
⊠	52.5	38.0	9.5	1.8	88.1
▲	56.4	39.0	4.5	3.0	28.0
★	59.7	35.7	4.7	2.1	27.9

GRADATION VALUES

	D60	D50	D30	D20	D10
●	0.126	0.10			
⊠	7.816	5.25	1.12	0.41	0.089
▲	7.355	5.64	2.39	0.79	0.263
★	10.251	6.93	2.79	1.30	0.368



Job No. **XL-2315** Date **July 28, 2005**
Hole No. **H-2-05** Sheet **1** of **1**
Project **Chehalis Western Trail Pedestrian Bridge**

Laboratory Summary

Washington State Department of Transportation

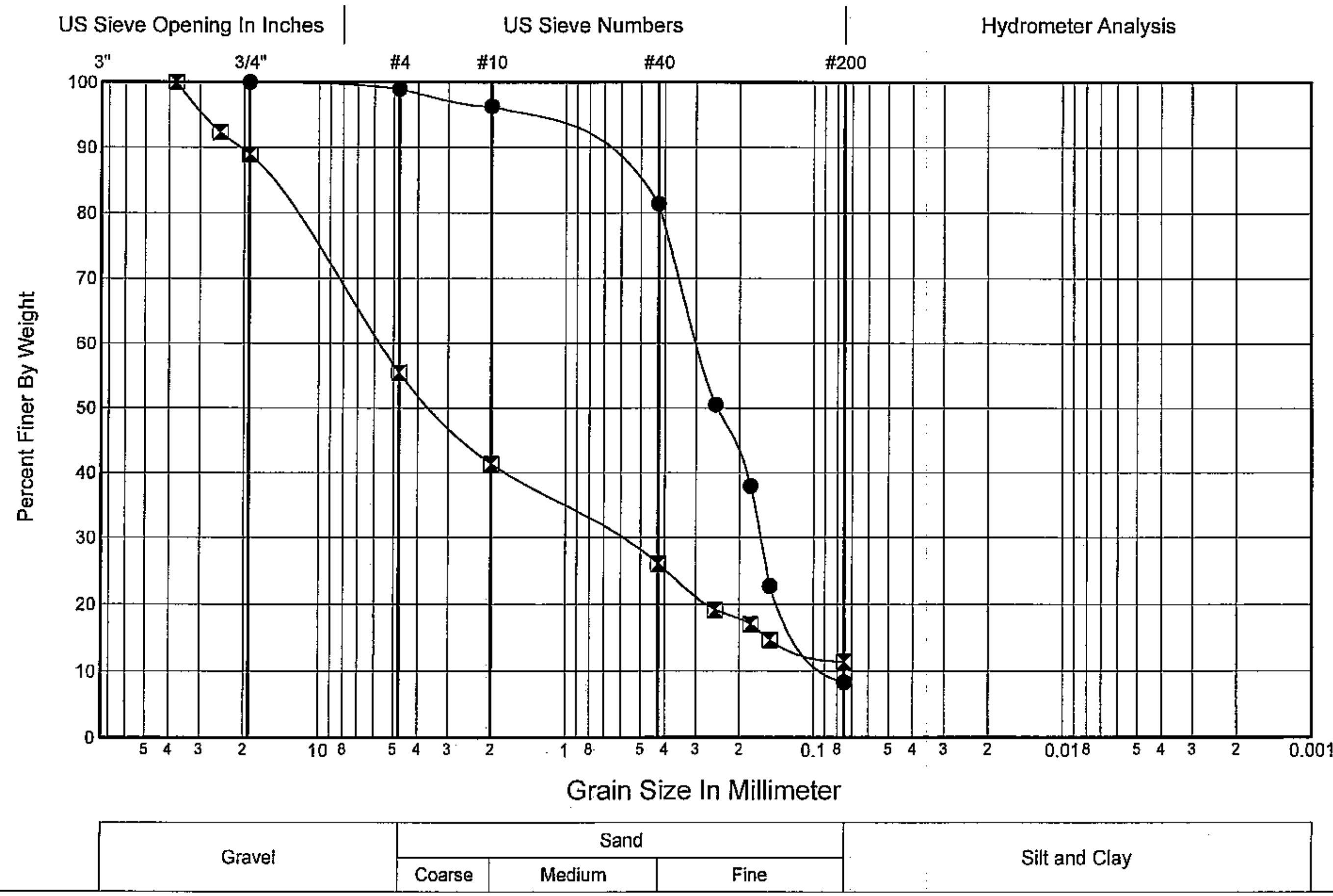
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	4.0	1.22	D-1	SP-SM	See Boring Log	POORLY GRADED SAND with SILT	20			
⊠	9.0	2.74	D-2	GW-GM	See Boring Log	WELL-GRADED GRAVEL with SILT and SAND	9			

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	1.0	90.6	8.3	1.1	3.6
⊠	44.6	44.1	11.4	1.2	101.8

GRADATION VALUES

	D60	D50	D30	D20	D10
●	0.294	0.25	0.16	0.13	0.081
⊠	5.736	3.40	0.63	0.27	



Job No.	XL-2315	Date	July 28, 2005
Hole No.	H-3-05	Sheet	1 of 1
Project	Chehalis Western Trail Pedestrian Bridge		

Laboratory Summary

Washington State Department of Transportation

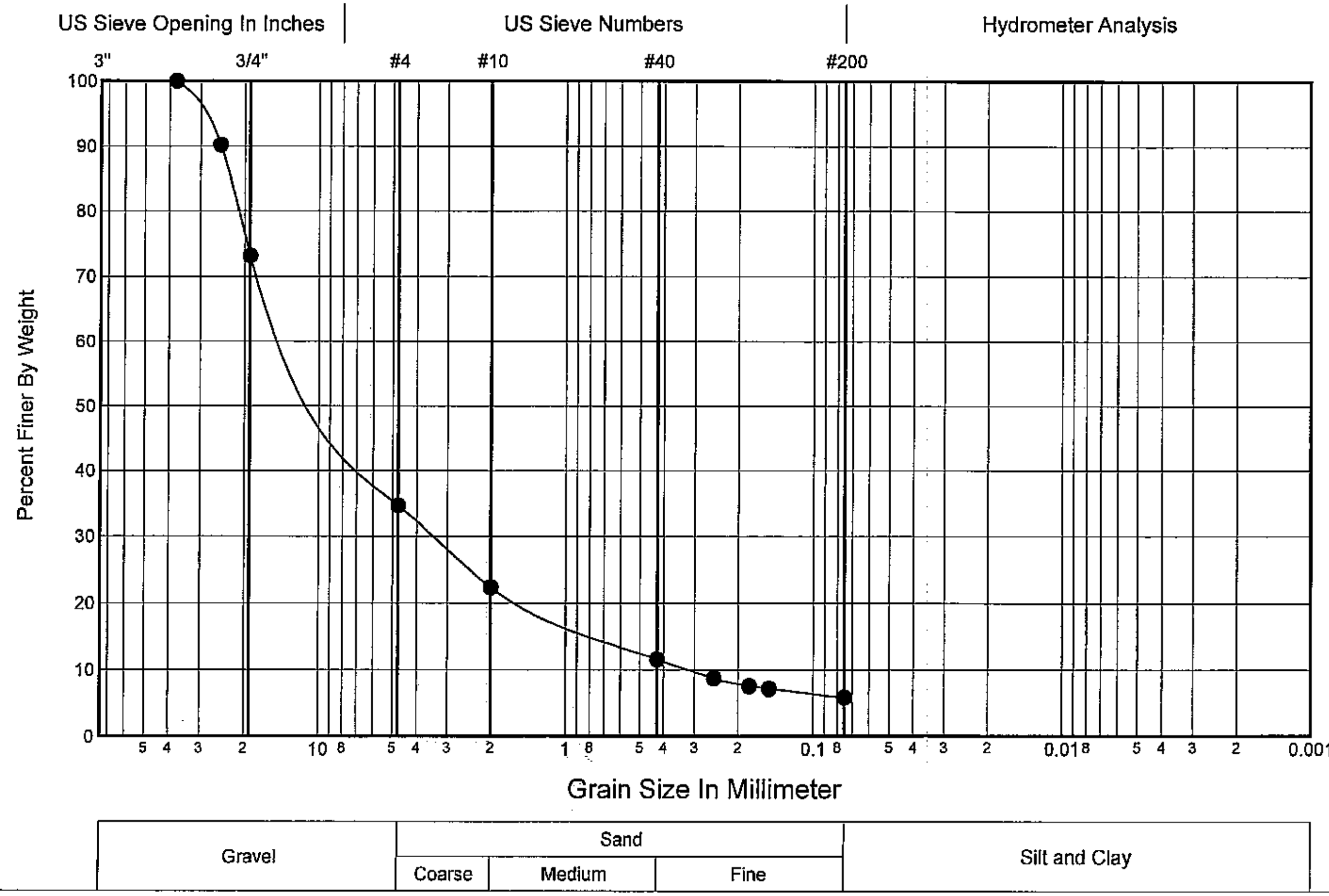
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	10.5	3.20	D-3	GP-GM	See Boring Log	POORLY GRADED GRAVEL with SILT and SAND	7			

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	65.4	28.8	5.9	3.1	37.3

GRADATION VALUES

	D60	D50	D30	D20	D10
●	11.843	8.26	3.43	1.43	0.318



WASHINGTON STATE
EPARTMENT OF TRANSPORTAT

MATERIALS ENGINEER
Materials Laboratory
P. O. Box 167, Olympia, WA 98504 (Mailing Address)
1655 So. 2nd Ave.
Tumwater, Washington 98504 (Shipping Address)

1

Place Olympia
Date 4/3/84

Dear Sir:

I have forwarded by today's Delivered the following Foundation Samples.

Contract or Job No. L-6941

Section Trosper Rd I/C to Martin way I/C
SR No. I-5 Sub-Section Weyerhaeuser RR U-Xing

Station & Offset WRR- 7+22 ℄

Hole # H-1

Lab No.	Drive #	Depth	Tube Position in Sampler	Clas.	Description
E 5345-1	D-1	2'-4'	H2O 18.1%	SM	
-2	D-2	7'-9'	24.6%	SM	
-3	D-3	12'-0" 13'-6"	9.5%	GW	
-4	D-4	17'-0" 18'-6"	7.0%	GW GM	
-5	D-5	22'-0" 22'-9"	10.5%	GW	
-6	D-6	27'-0" 28'-6"	11.0%	GW	
-7	D-7	32'-0" 33'-6"	14.8%	GW	
-8	D-8	37'-38'			Same as 5345-7
-9	D-9	42'-0" 42'-9"			Same as 5345-7

1 copy with samples
1 copy to addressee

Yours very truly,
Over 2
Inspector.

DOT FORM 351-002 REVISED 2/80

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

MATERIALS ENGINEER
Materials Laboratory
P. O. Box 167, Olympia, WA 98504 (Mailing Address)
1655 So. 2nd Ave.
Tumwater, Washington 98504 (Shipping Address)

2

Place Olympia

Date

Dear Sir:

I have forwarded by today's .... the following Foundation Samples.

Contract or Job No. 1-6941 Section SR No. I-5 Sub-Section Weyerhaeuser RR U-Xing

Station & Offset WRR 7+22 Hole # H-1

E Lab No.	Drive #	Depth	Tube Position in Sampler	Clas.	Description
5345-10	D-10	47'-0" 48'-6"	11.4%	GP	

1 copy with samples
1 copy to addressee

Yours very truly,

C.N. Sevill

Inspector.

DOT FORM 351-002 REVISED 2/80

SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5345-1 JOB NO. L-6941 HOLE NO. H-1
DATE 4-13-84 OPERATOR JM

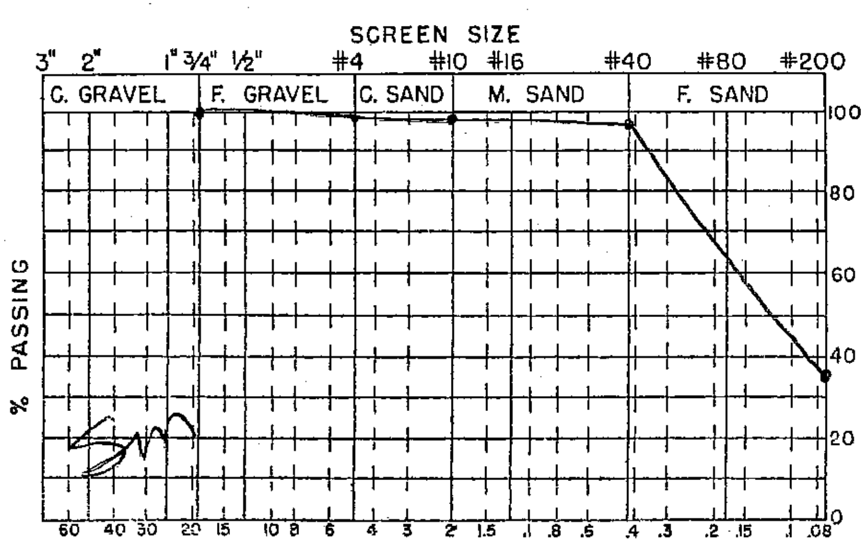
SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: BRN., MOIST, V. SILTY, F-M SAND, W/TRACE OF ORGANIC

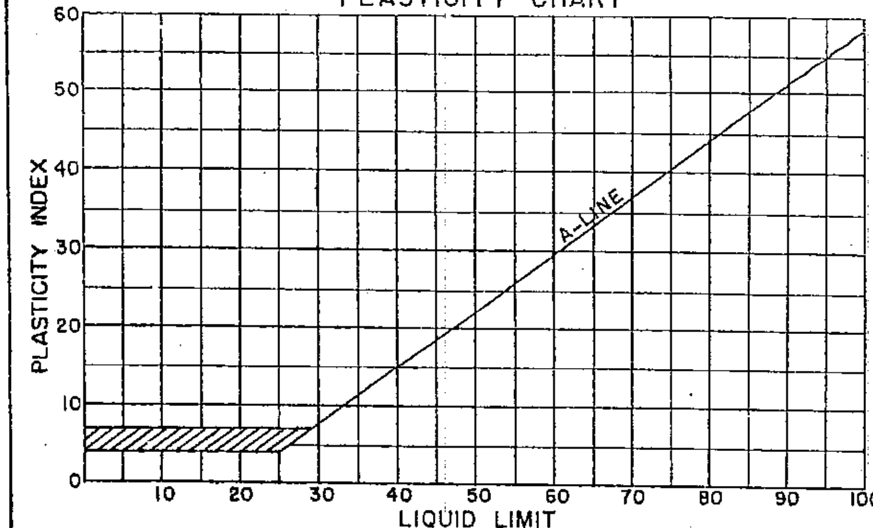
34.90 41.22 H_2O = 18.1 %

SIEVE ANALYSIS		% Passing	199.7	
		-40	121.75	97.1
-3/4	.30	100	-80	
-4	.20	99.8	-140	
-10	5.20	99.7	-200 72.25	36.2

GRAIN SIZE CURVE

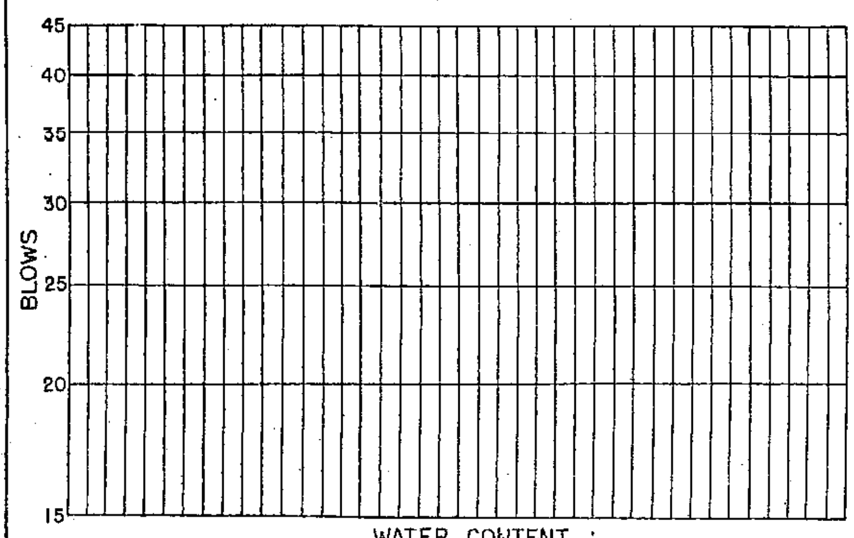


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=

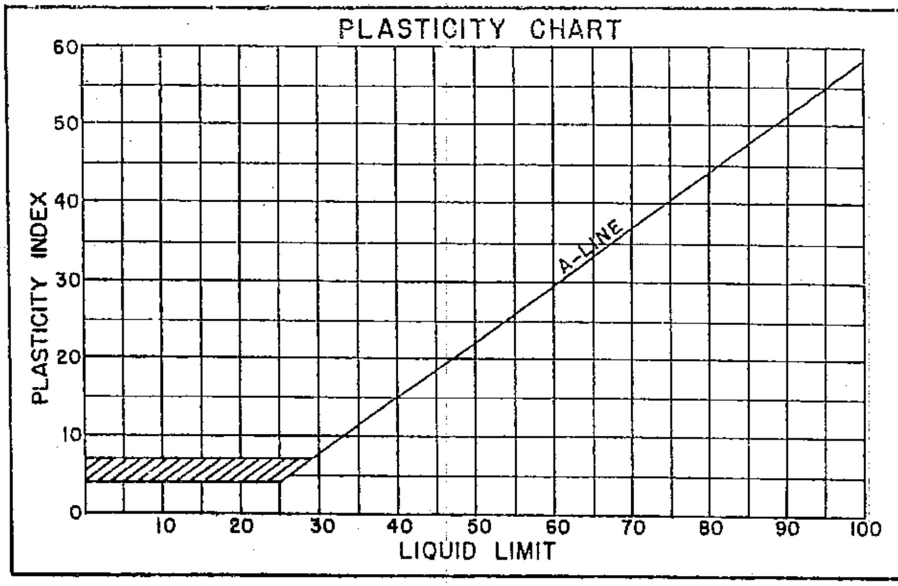


SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5345-2 JOB NO. L-6941 HOLE NO. H-1
DATE 4-13-84 OPERATOR Jm

SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

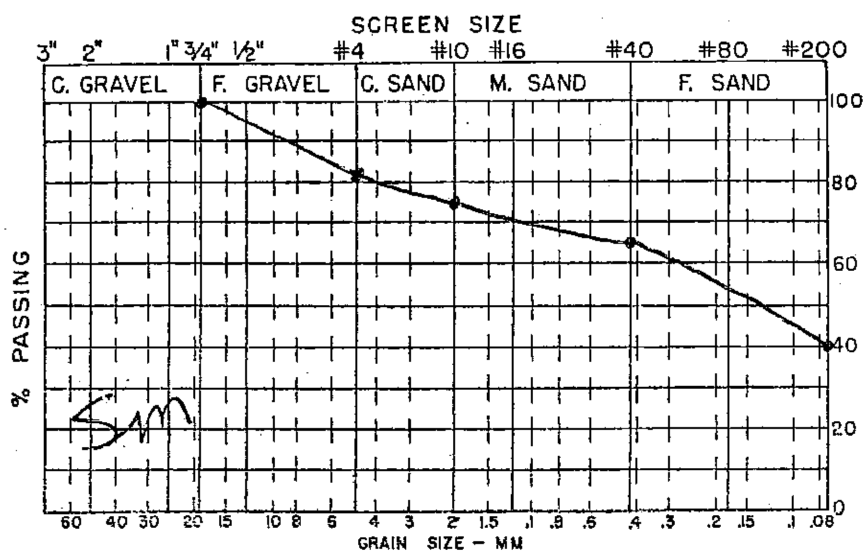
DESCRIPTION: BRN.-GREY, MOIST, V. SILTY ~~F-C SAND~~ FINE GRAVELLY, F-C SAND



50.0 62.32 H2O= 24.6 %

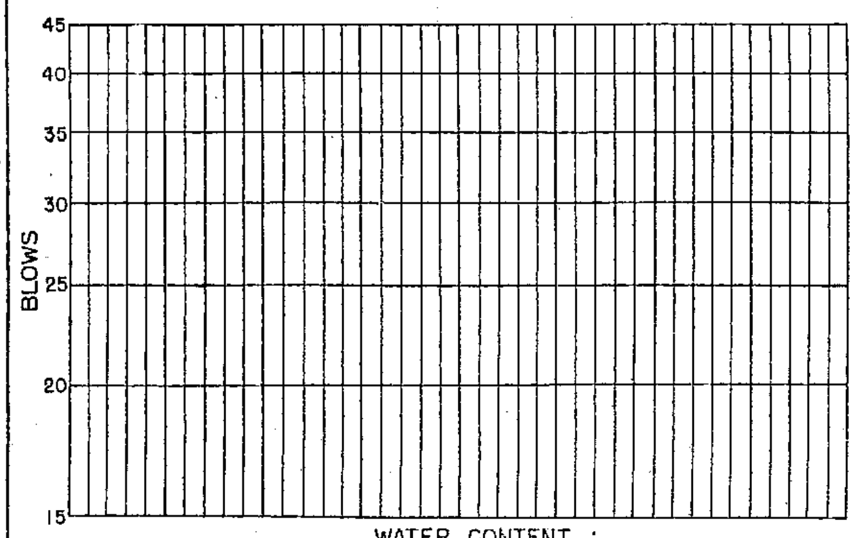
SIEVE ANALYSIS			% Passing 233.9		
			-40	58.08	65.0
-3/4	44.92	100	-80		
-4	14.55	80.8	-140		
-10	22.30	74.6	-200	94.05	40.2

GRAIN SIZE CURVE



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
$\% H_2O$					PL=
Blows					PI=



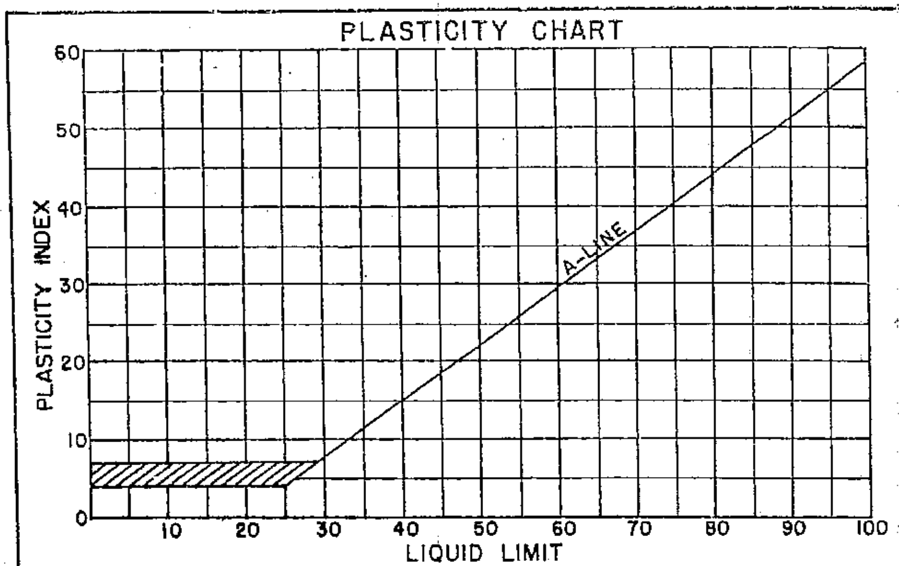
SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5345-3 JOB NO. L-6941 HOLE NO. A-1
DATE 4-13-84 OPERATOR JM

SOIL FIELD IDENTIFICATION

TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY, DRY, SL. SILTY, F-C SANDY, 1" ANG & RND. GRAVEL

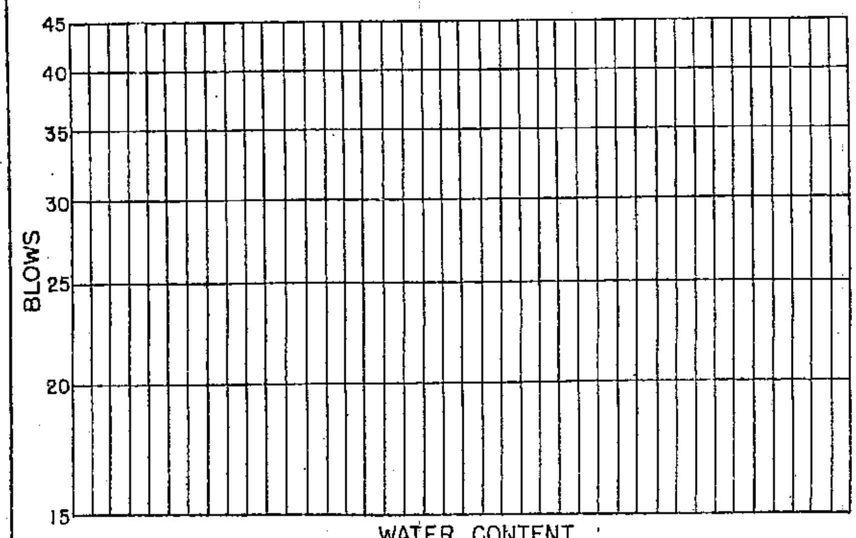


59.5 65.18 H_2O = 9.5 %

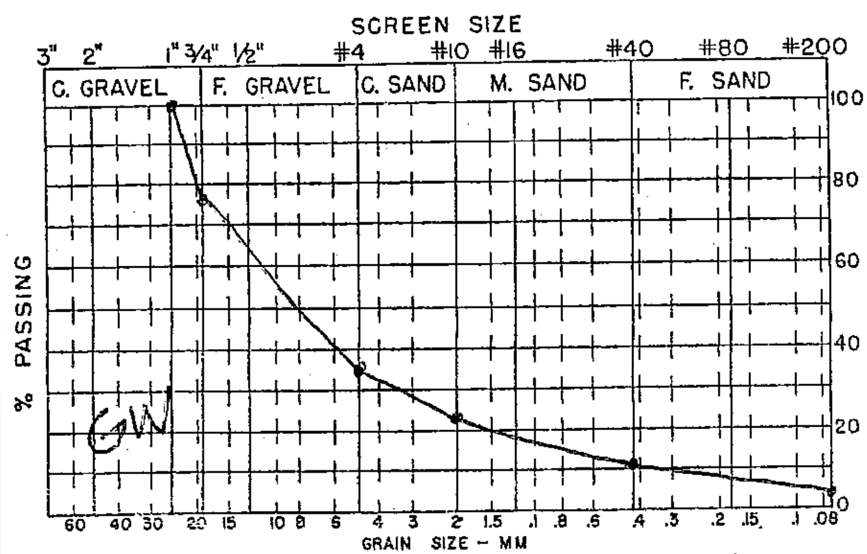
SIEVE ANALYSIS		% Passing	235.10		
-1"	55.87	100	-40	13.65	10.4
-3/4	97.20	76.2	-80		
-4	27.30	34.9	-140		
-10	30.25	23.3	-200	10.83	4.60

LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



GRAIN SIZE CURVE



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 53454 JOB NO. L-6941 HOLE NO. H-1
DATE 4-13-84 OPERATOR JM

SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

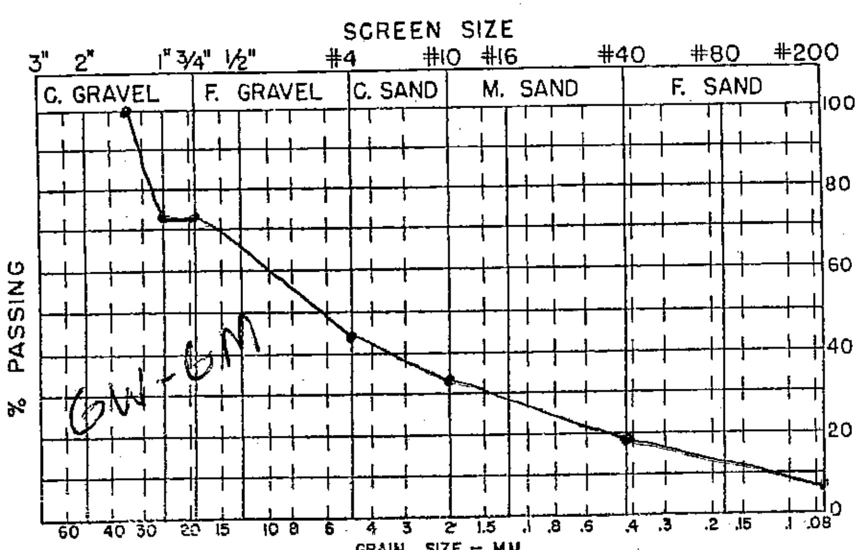
DESCRIPTION: GREY-BRN, DRY, SILTY, F-C SANDY, 1½"- S. RND. GRAVEL

71.30 76.30 H20 = 7.0 %

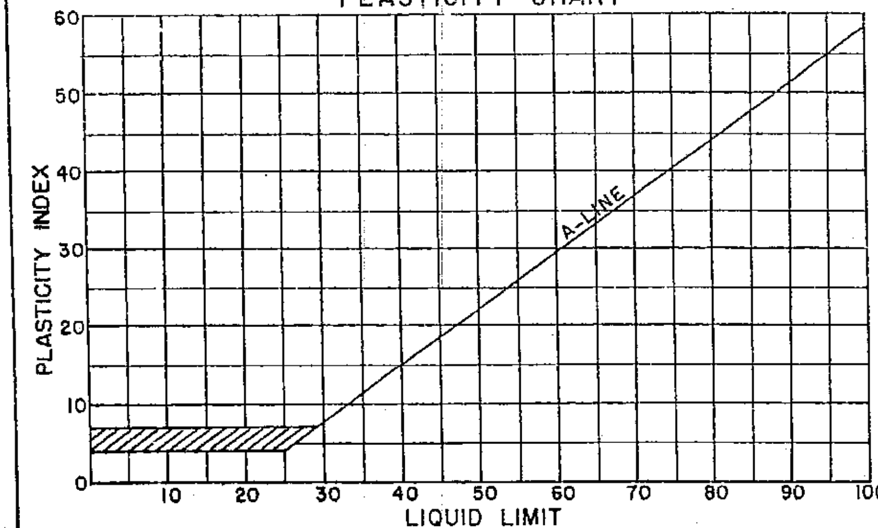
100

-1½" 77.0	SIEVE ANALYSIS		% Passing 298.60		
-1"	0	74.2	-40	32.45	18.4
-¾	89.85	74.2	-80		
-4	32.05	44.1	-140		
-10	44.63	33.4	-200	22.62	7.6

GRAIN SIZE CURVE

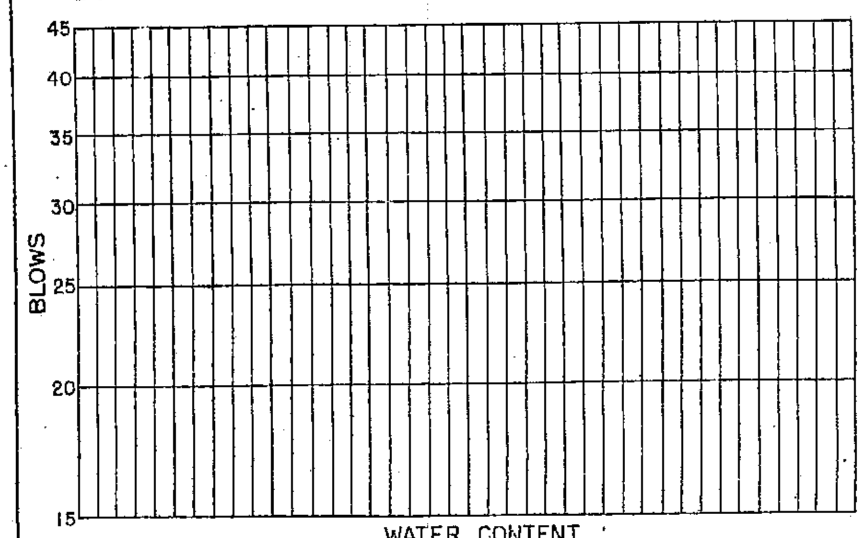


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



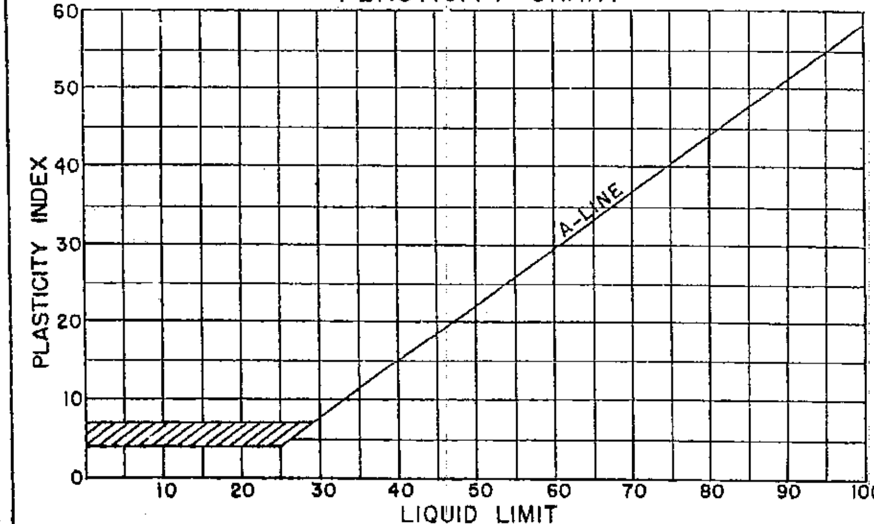
SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5345-5 JOB NO. L-6941 HOLE NO. H-1
DATE 4-13-84 OPERATOR Jm

SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY, MOIST, SL SILTY, F-C SANDY, 1½"- S! RND. GRAVEL

PLASTICITY CHART



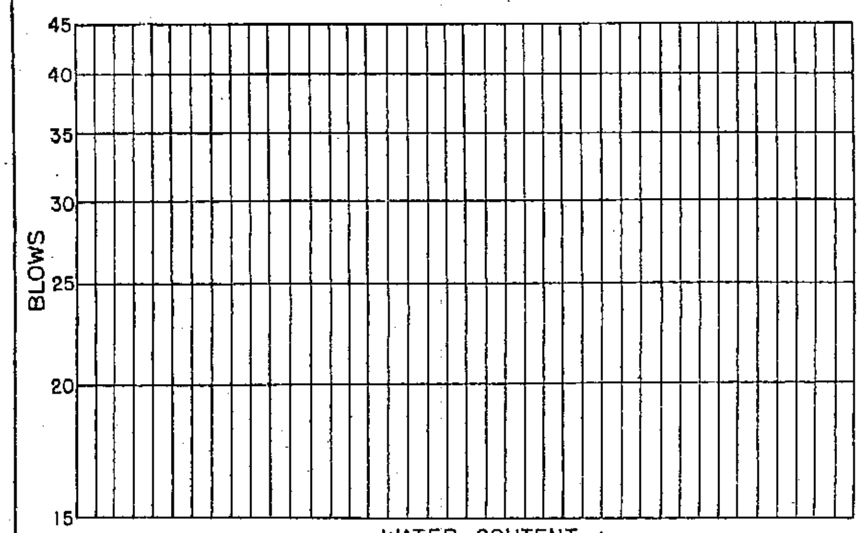
52.10 57.58 H2O-10.5%

100

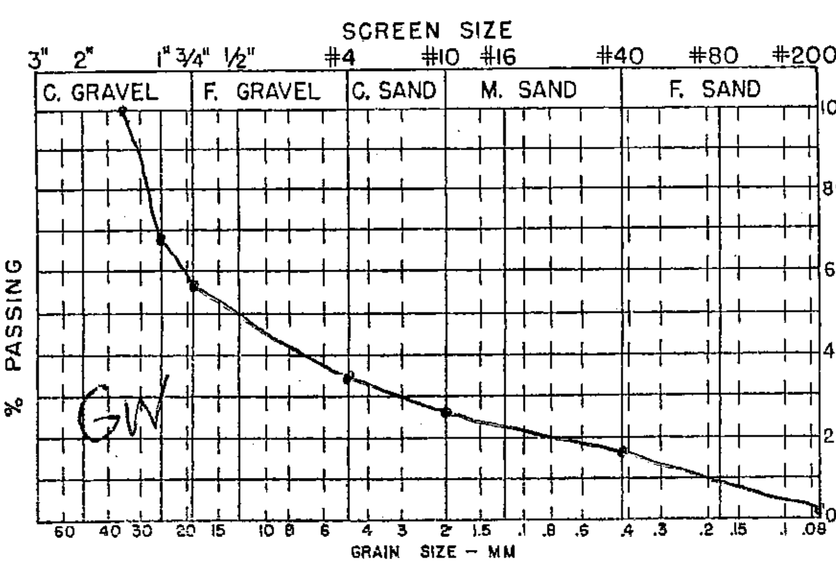
-1½" 90.30	SIEVE ANALYSIS	% Passing	287.55
-1" 32.60	68.6	-40 40.20	16.4
-¾ 64.82	57.3	-80	
-4 21.90	34.7	-140	
-10 30.70	27.1	-200 7.03	2.4

LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



GRAIN SIZE CURVE



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

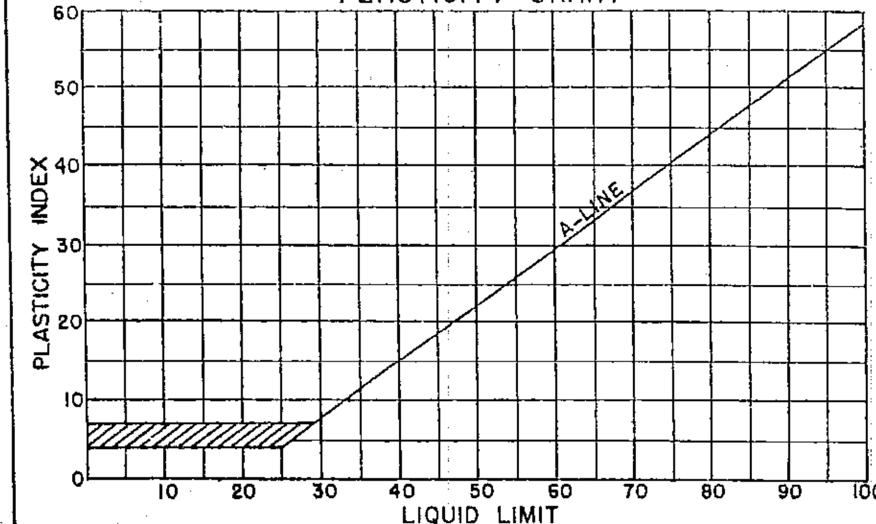
SAMPLE NO. 5345-6 JOB NO. L-6941 HOLE NO. H-1
DATE 4-13-84 OPERATOR Jm

SOIL FIELD IDENTIFICATION

TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY-BRN., MOIST, SL. SILTY, F-C SANDY, 1½"- ANG & S. RND. GRAVEL.

PLASTICITY CHART

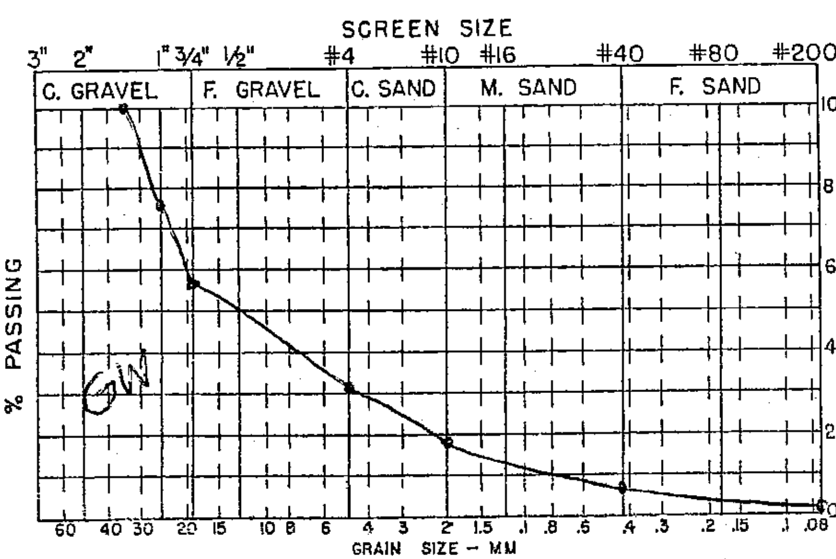


63.85 70.85 H_2O = 11.0 %

100

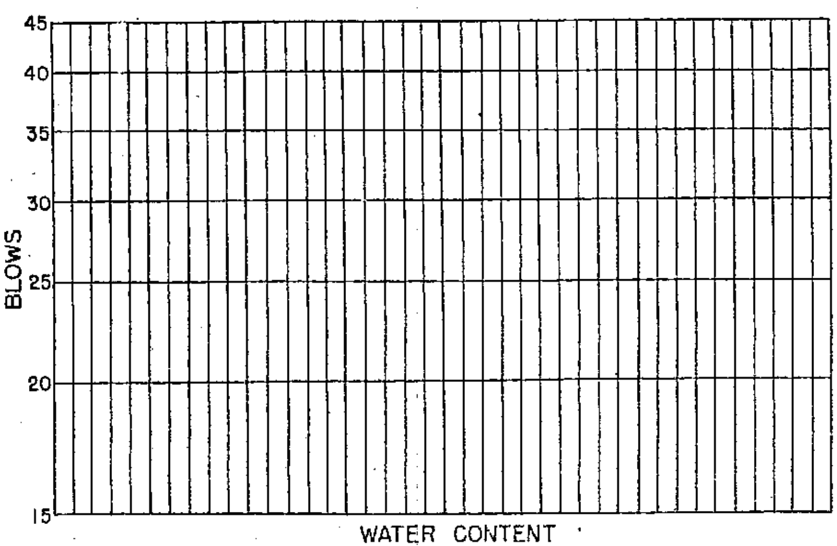
-1½" 69.65 SIEVE ANALYSIS		% Passing 286.20	
-1" 5350	75.7	-40 15.20	6.8
-¾ 72.52	57.0	-80	
-4 38.68	31.6	-140	
-10 32.45	18.1	-200 4.20	1.5

GRAIN SIZE CURVE



LIQUID LIMIT DETERMINATION

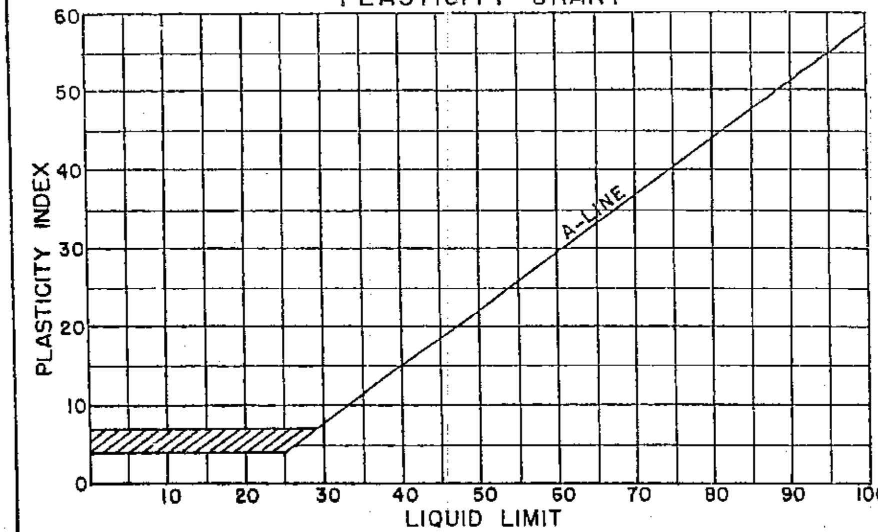
	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5345-7 JOB NO. L-6941 HOLE NO. H-1			
DATE 4-13-84 OPERATOR JM			
SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			
DESCRIPTION: GREY-BRN., MOIST, F-C			
SANDY, 1½" - S. RND. GRAVEL			

PLASTICITY CHART



57.70 66.25 H2O=14.3 %

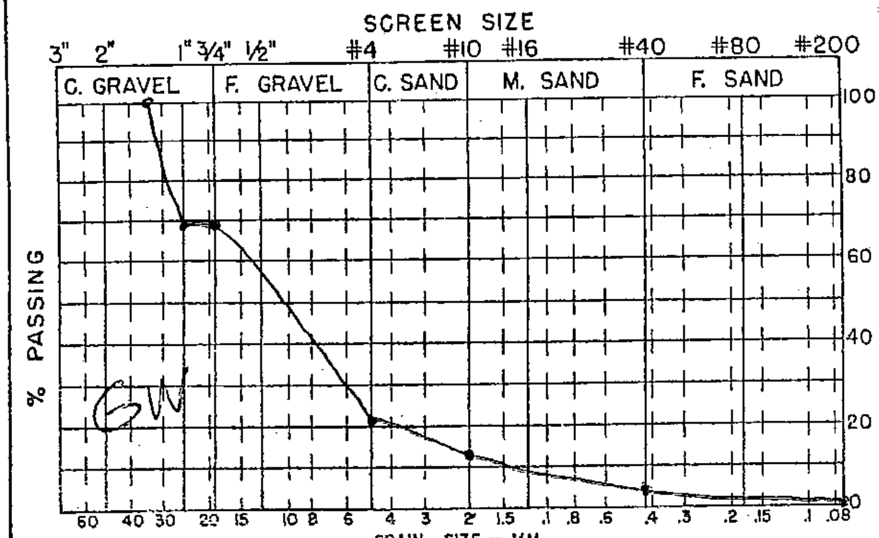
100

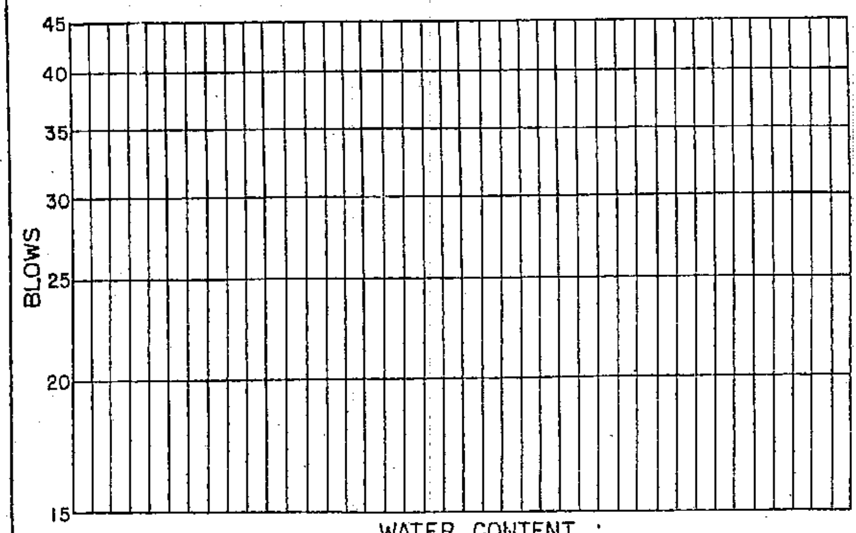
SIEVE ANALYSIS		% Passing	239.82
-1½"	73.30		
-1"	0 69.4 ~~68.4~~	-40	9.68 4.3
-¾	115.40 69.4 ~~69.4~~	-80	
-4	21.72 21.3 ~~18.6~~	-140	
-10	19.12 12.3 ~~9.2~~	-200	.60 0.3

LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=

GRAIN SIZE CURVE





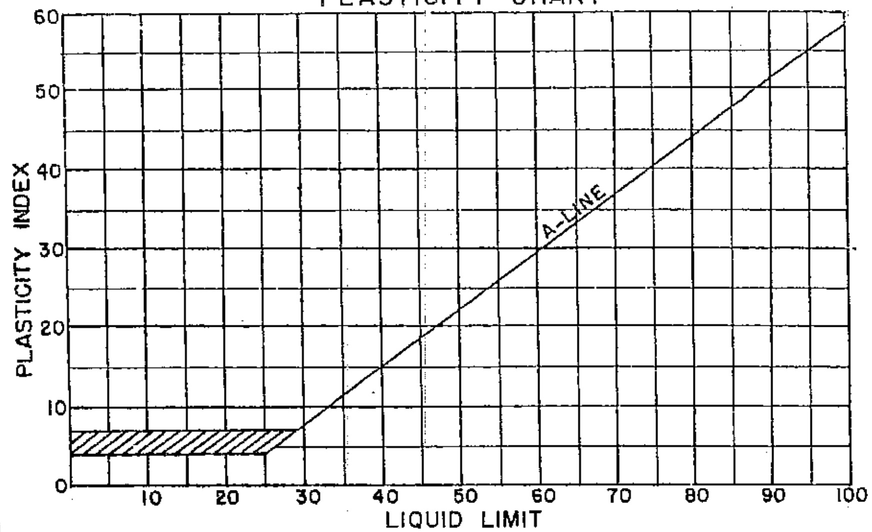
SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 534510 JOB NO. L-6941 HOLE NO. H-1
DATE 4-13-84 OPERATOR Jm

SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY, MOIST, SL. SILTY, F-C SANDY, 1½" - S.RND. GRAVEL

PLASTICITY CHART



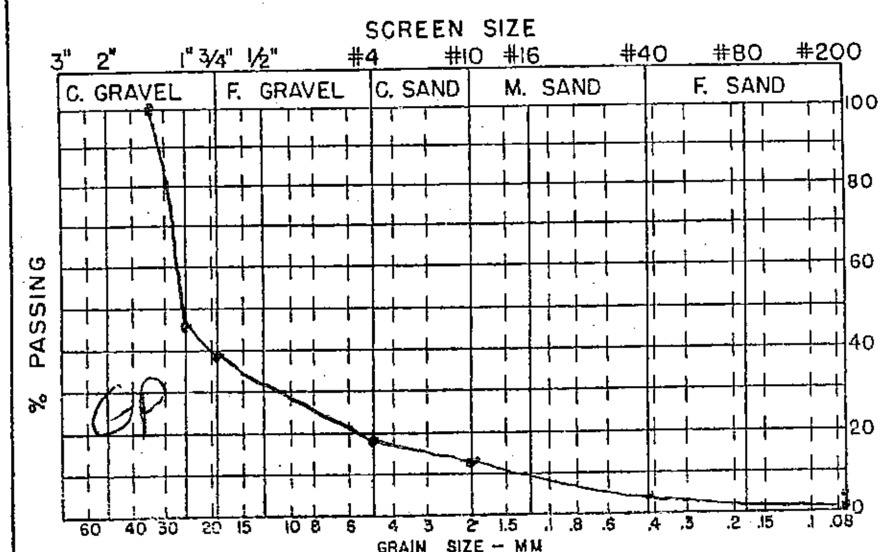
58.40 65.05 H_2O = 11.4 %

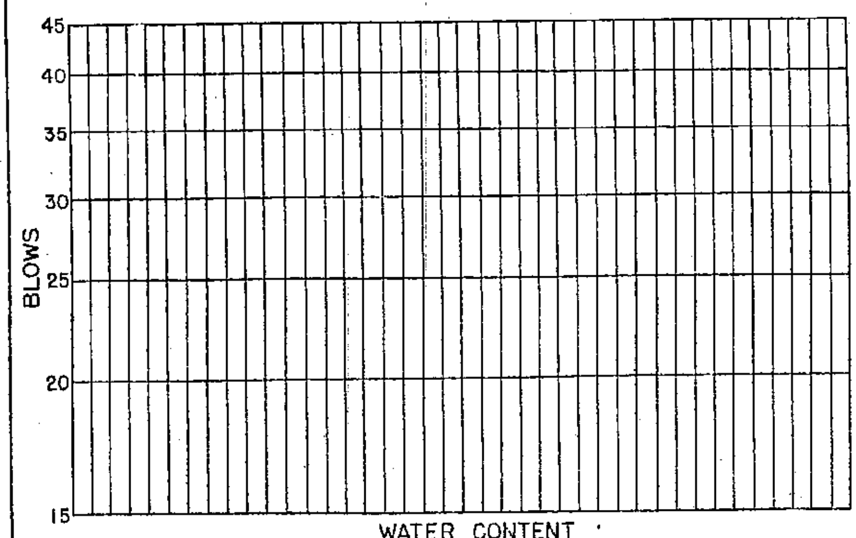
SIEVE ANALYSIS			% Passing	294.85	
-1½"	160.20	100			
-1"	19.15	45.7	-40	11.0	4.9
-¾	61.12	39.2	-80		
-4	17.70	18.4	-140		
-10	22.33	12.4	-200	3.35	1.1

LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=

GRAIN SIZE CURVE





WASHINGTON STATE
EPARTMENT OF TRANSPORTAT. N

MATERIALS ENGINEER
Materials Laboratory
P. O. Box 167, Olympia, WA 98504 (Mailing Address)
1655 So. 2nd Ave.
Tumwater, Washington 98504 (Shipping Address)

Place Olympia
Date 4/3/84

Dear Sir:

I have forwarded by today's Delivered the following Foundation Samples.

Contract or Job No. L-6941 Section Trosper Rd. I/C To martin way I/C
SR No. Sub-Section weyerhaeuser R.R u-xing

Station & Offset WRR. 9+42 Hole # H-2

E Lab No.	Drive #	Depth	Tube Position in Sampler	Clas.	Description
5346-1	D-1	2'-4'	H2O 16.1%	SP SM	
-2	D-2	7'-9'	9.3%	GW GM	
-3	D-3	12'-0" 13'-6"	9.7%	GP GM	
-4	D-4	17'-0" 17'-6"	9.7%	GP	
-5	D-5	22'-0" 22'-6"	8.1%	GW	
-6	D-6	27'-28'	13.8%	SP	
-7	D-7	32'-33'			Same AS 5346-6
-8	D-8	37'-38'			Same AS 5346-6
-9	D-9	42'-43'	13.8%	GW	

1 copy with samples
1 copy to addressee

Yours very truly,
C.N. Savill
Inspector.

DOT FORM 351-002 REVISED 2/80

SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5346-1 JOB NO. L-6941 HOLE NO. H-2
DATE 4-13-84 OPERATOR JM

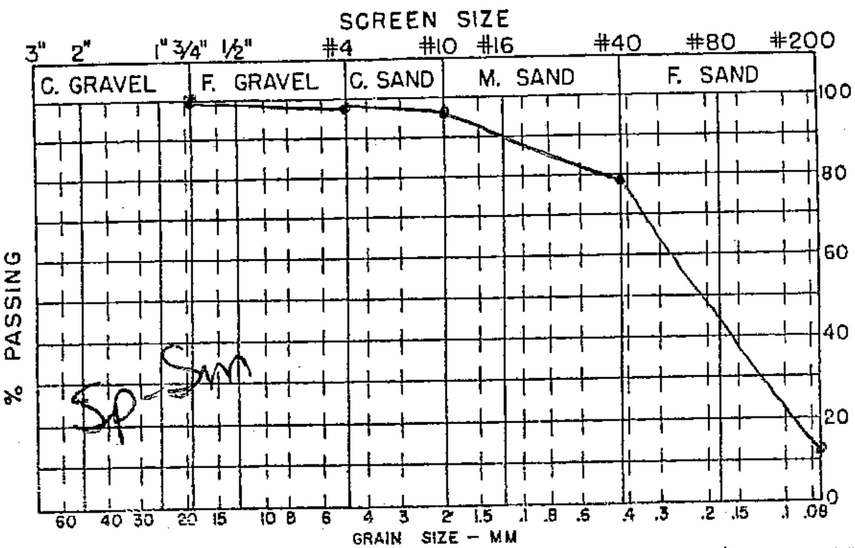
SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: BRN., MOIST, SILTY, F. GRAVELLY, F-C SAND

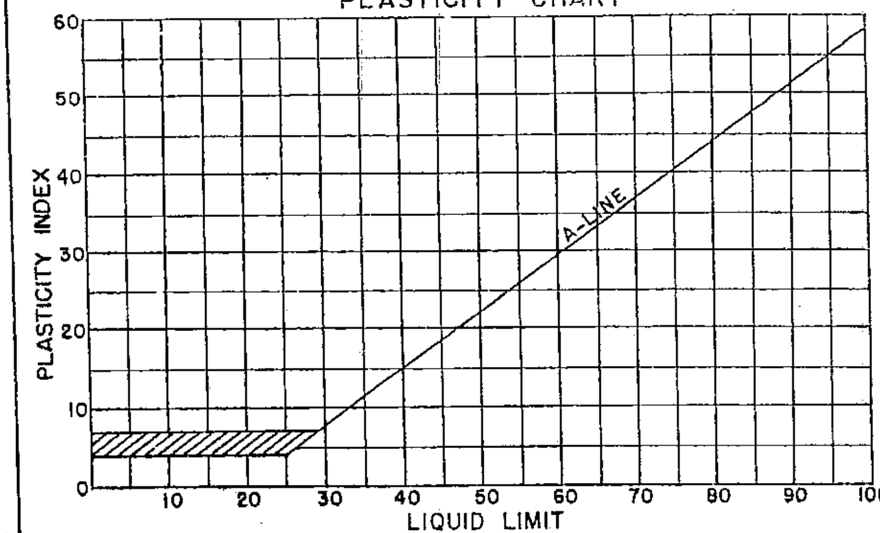
26.72 31.02 H20=16.1 %

SIEVE ANALYSIS		% Passing 219.08	
		-40 148.08	79.2
-3/4 4.18	100	-80	
-4 2.62	98.1	-140	
-10 38.84	96.9	-200 25.36	11.6

GRAIN SIZE CURVE

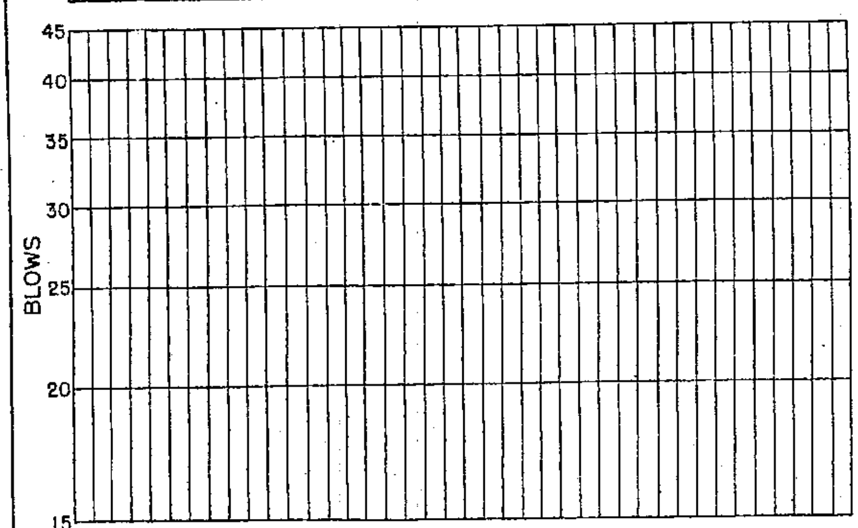


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5346-2 JOB NO. L-6941 HOLE NO. H-2
DATE 4-13-84 OPERATOR JM

SOIL FIELD IDENTIFICATION

TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

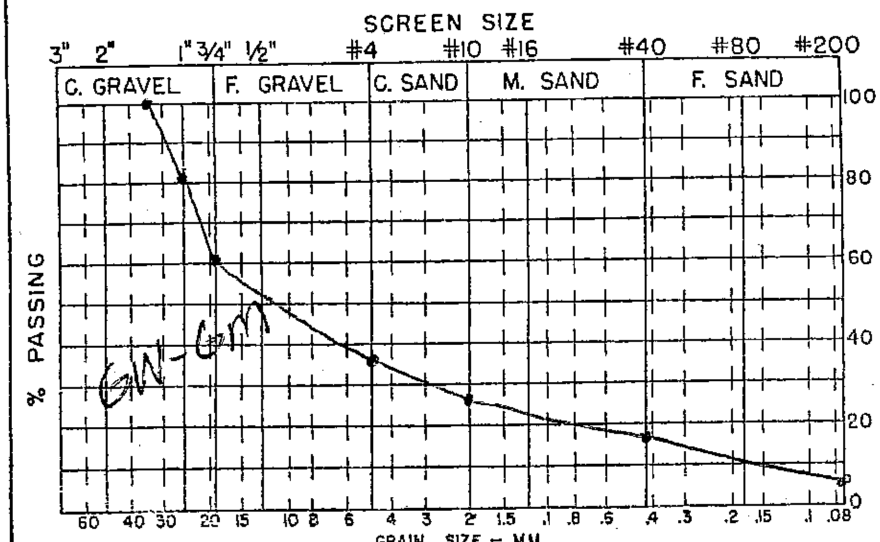
DESCRIPTION: GREY-BRN., DRY, SILTY, F-C SANDY, 1 1/2"- ANG. & SIRND. GRAVEL

53.50 58.50 H20 = 9.3 %

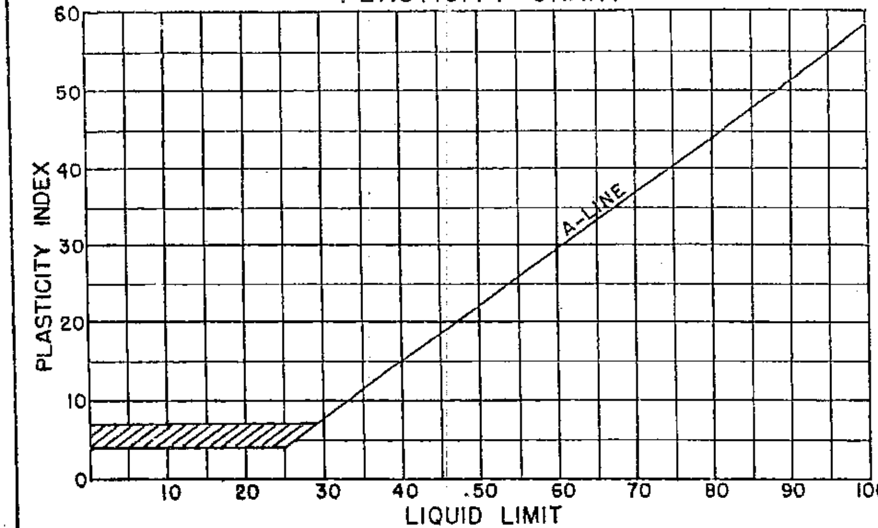
100

-1 1/2"	42.82	SIEVE ANALYSIS	% Passing	233.85	
-1"	48.95	81.7	-40	23.85	17.0
-3/4	57.60	60.8	-80		
-4	20.60	36.1	-140		
-10	24.18	27.3	-200	15.85	6.8

GRAIN SIZE CURVE

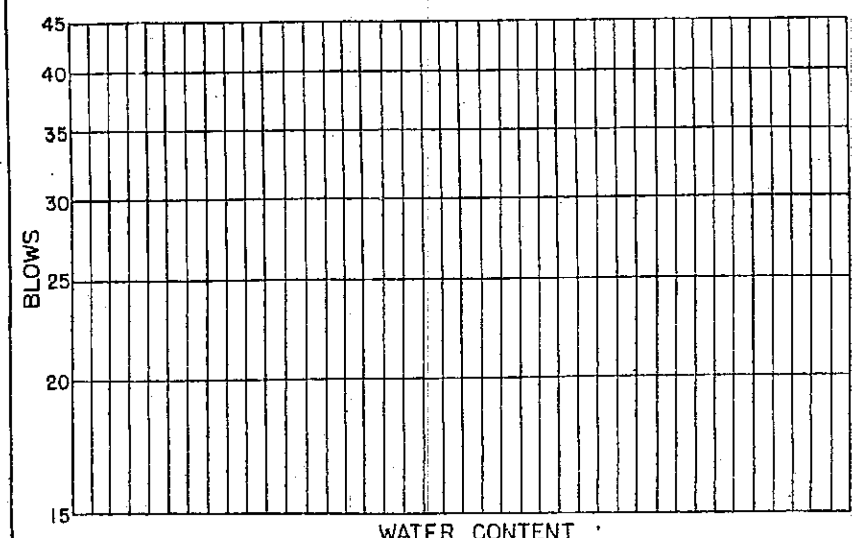


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

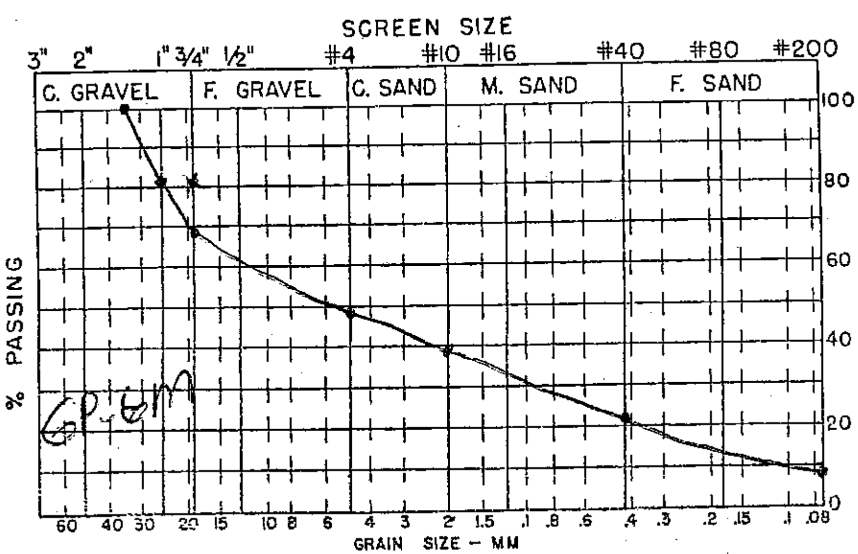
SAMPLE NO. 5346-3 JOB NO. L-6941 HOLE NO. H-2			
DATE 4-13-84 OPERATOR Jm			
SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			
DESCRIPTION: BRN., DRY, SILTY, F-C SANDY, 1½"- S. RND. GRAVEL			

70.90 77.80 H_2O = 9.7 %

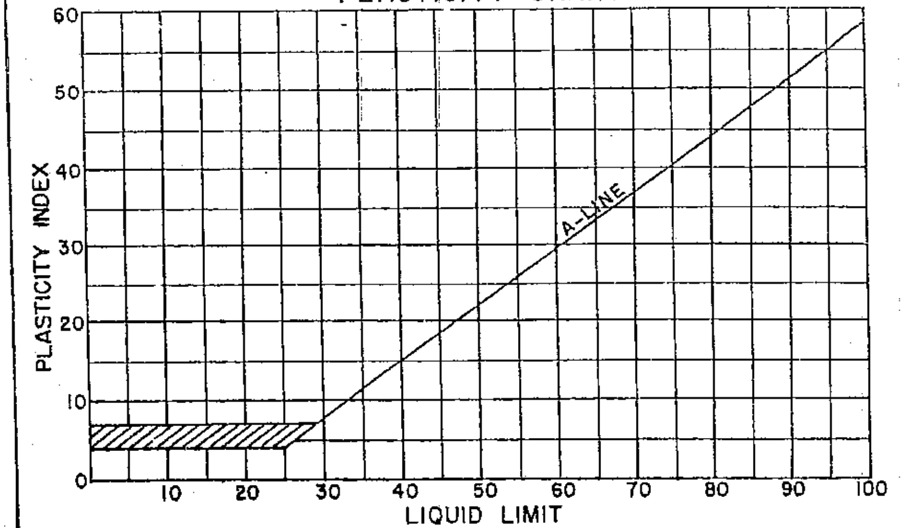
100

-1½"	33.68	SIEVE ANALYSIS	% Passing	100	42
-1"	23.45	81.3	-40	23.75	21.6
-¾	35.98	68.3	-80		
-4	16.38	49.4	-140		
-10	32.0	39.3	-200	15.18	8.4

GRAIN SIZE CURVE

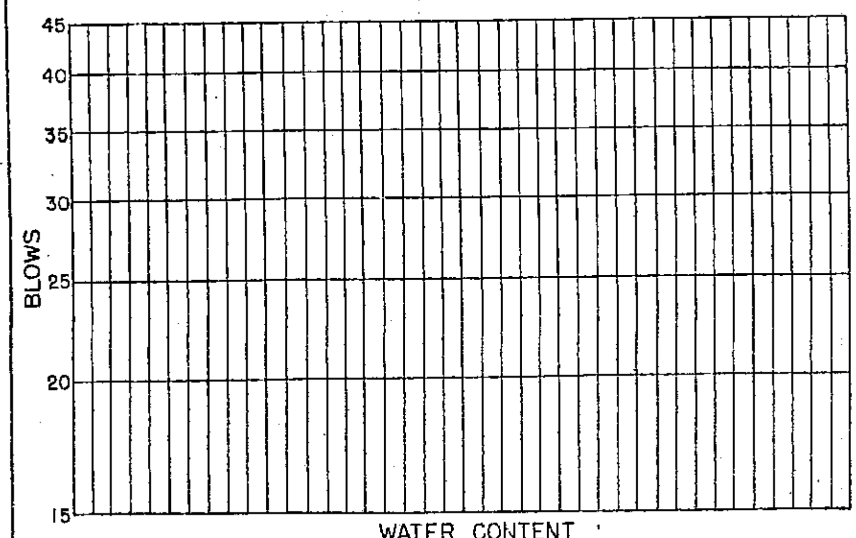


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



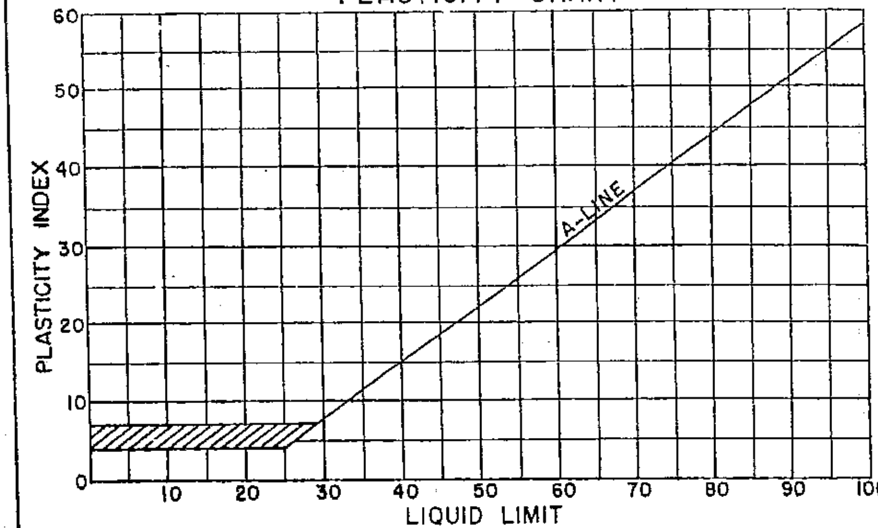
SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5346-4 JOB NO. L-6941 HOLE NO. H-2
DATE 4-13-84 OPERATOR Jm

SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY-BRN., DRY, SL. SILTY, F-C SANDY, 1½" ANG. & SRND. GRAV.

PLASTICITY CHART

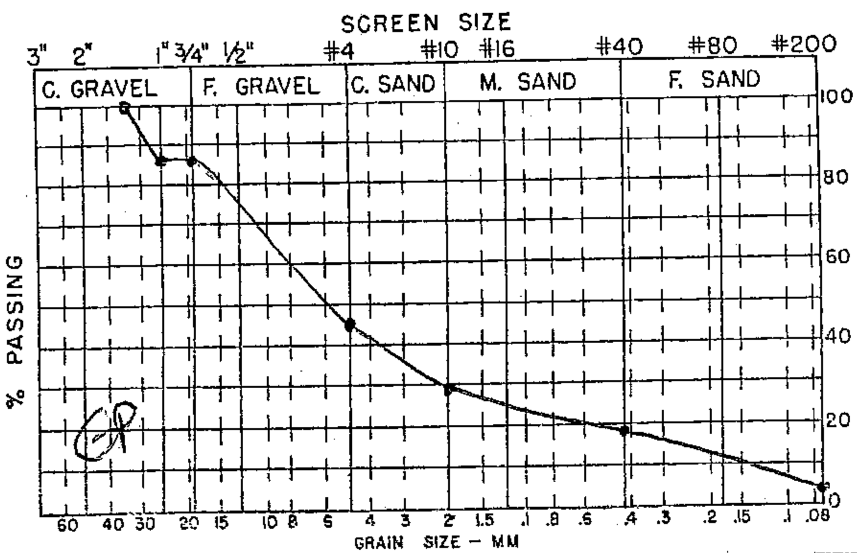


61.80 67.80 H_2O = 9.7 %

100

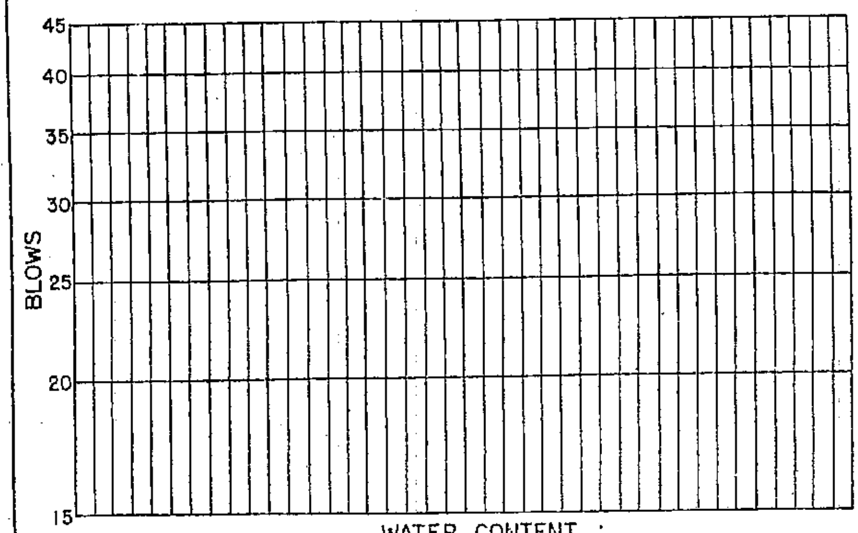
-1½" 31.50 SIEVE ANALYSIS		% Passing 247.20	
-1" 0	87.3	-40 36.92	17.6
-¾ 105.48	87.3	-80	
-4 38.60	44.6	-140	
-10 28.12	29.0	-200 6.58	2.7

GRAIN SIZE CURVE



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

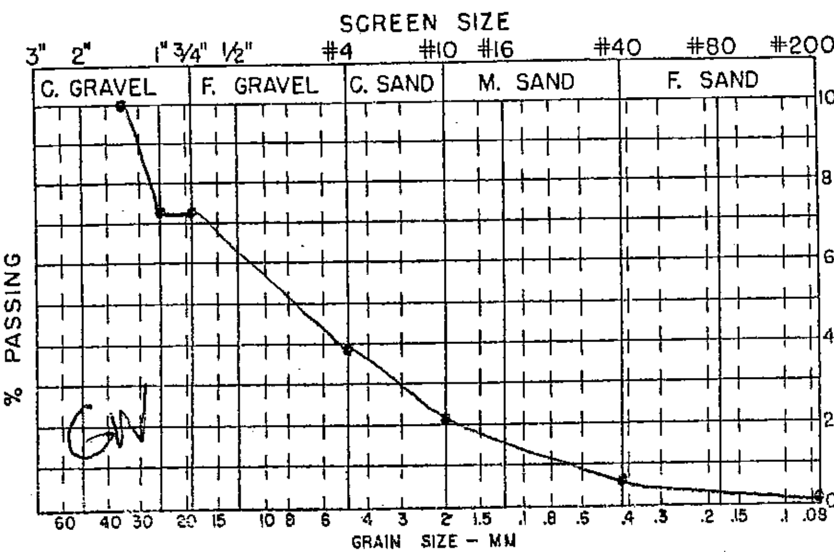
SAMPLE NO. 5346-5 JOB NO. L-6941 HOLE NO. H-2			
DATE 4-13-84 OPERATOR JM			
SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			
DESCRIPTION: GREY, DRY, SL. SILTY, F-C SANDY, 1½" S. RND. GRAVEL			

65.5 70.78 H_2O= 8.1 %

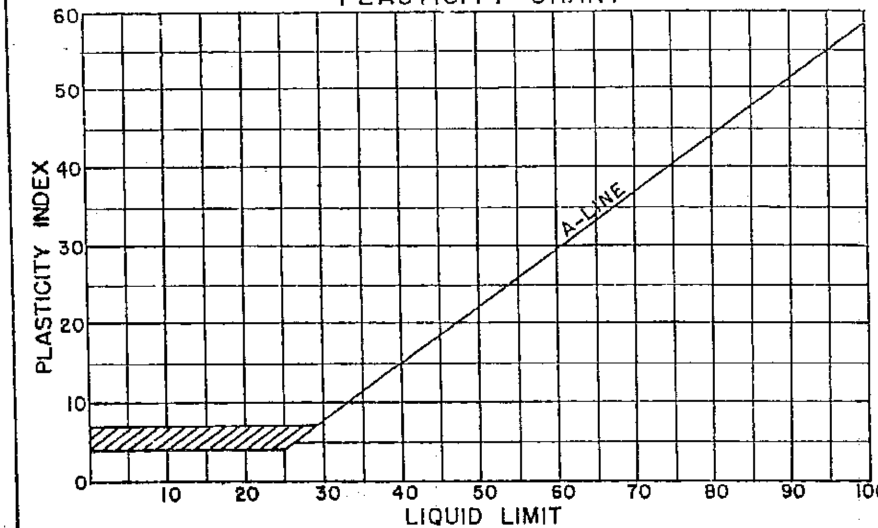
100

-1½"	72.42 SIEVE ANALYSIS	% Passing		264.30	
-1"	0	72.6	-40	12.0	6.1
-¾	88.0	72.6	-80		
-4	47.85	39.3	-140		
-10	40.28	21.4	-200	4.15	1.6

GRAIN SIZE CURVE

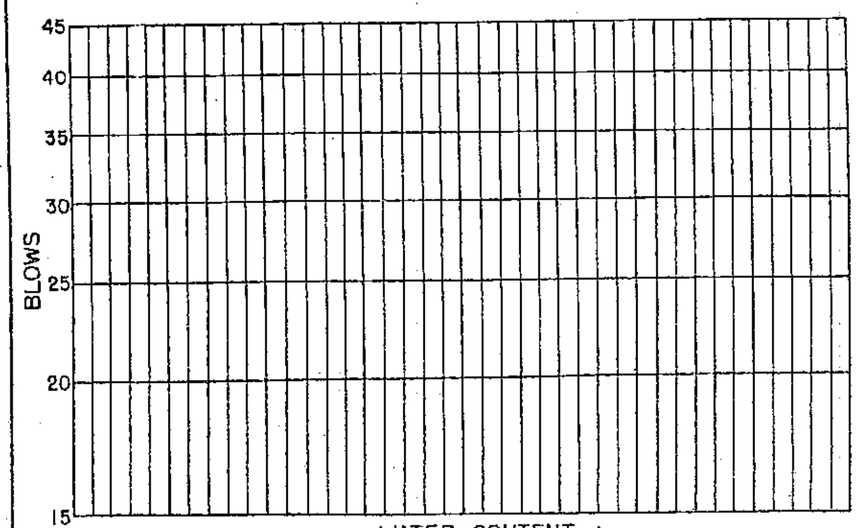


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

SAMPLE NO. 5346-6 JOB NO. L-6941 HOLE NO. H-2
DATE 4-13-84 OPERATOR Jm

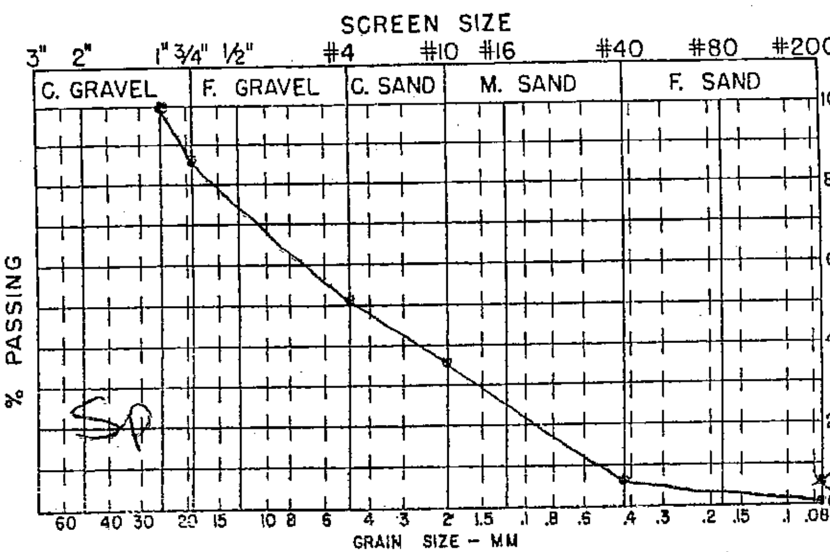
SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY, MOIST, F-C GRAVELLY F-C SAND

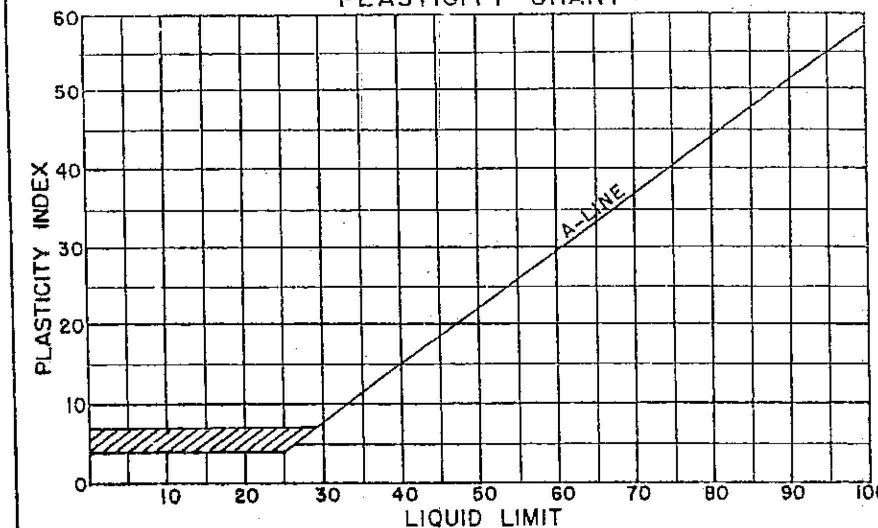
110.40 125.65 H2O = 13.8 %

SIEVE ANALYSIS		% Passing 506.01			
-1"	71.60	100	-40	32.38	7.2
-3/4	174.98	85.9	-80		
-4	76.70	51.3	-140		
-10	146.55	36.1	-200	3.80	0.8

GRAIN SIZE CURVE

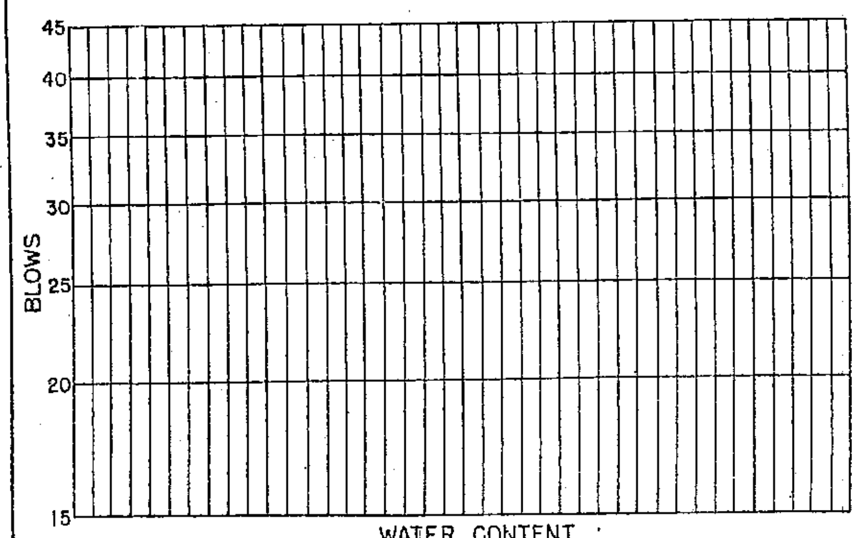


PLASTICITY CHART



LIQUID LIMIT DETERMINATION

	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



SOIL CLASSIFICATION AND IDENTIFICATION WORKSHEET

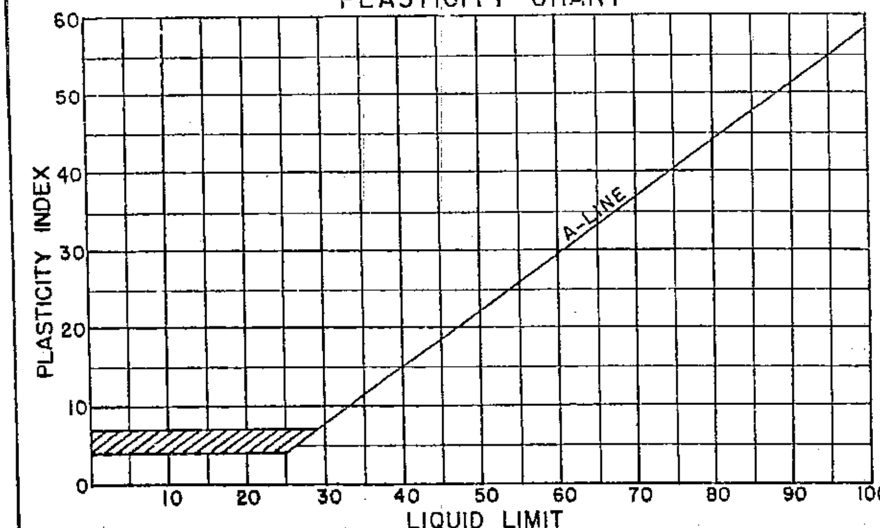
SAMPLE NO. 5346-9 JOB NO. L-6941 HOLE NO. H-2

DATE 4-13-84 OPERATOR Jm

SOIL FIELD IDENTIFICATION			
TEST	SAND	SILT	CLAY
VISUAL	✓	✓	
DRIED CAST			
DILATANCY			
BITE			
TOUGHNESS			

DESCRIPTION: GREY, MOIST, F-C SANDY 1 1/2" - SUB RND. GRAV.

PLASTICITY CHART

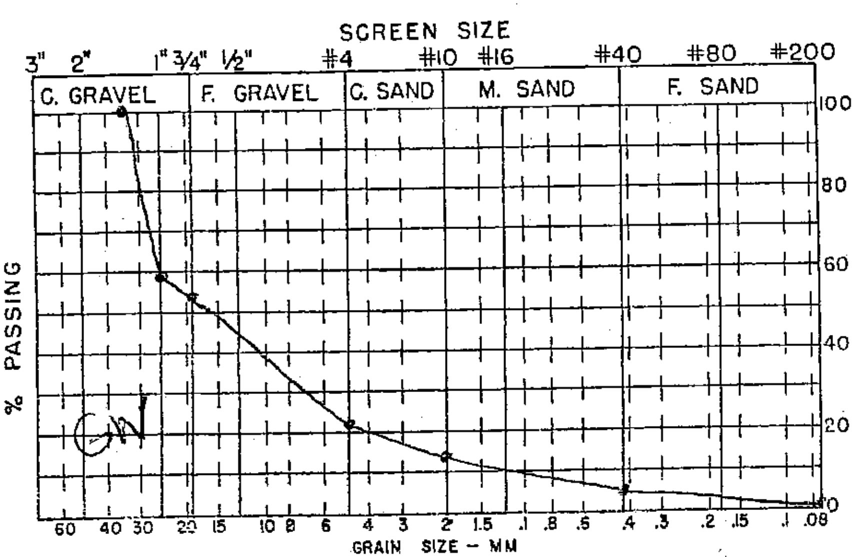


50.78 57.80 H_2O = 13.8 %

100

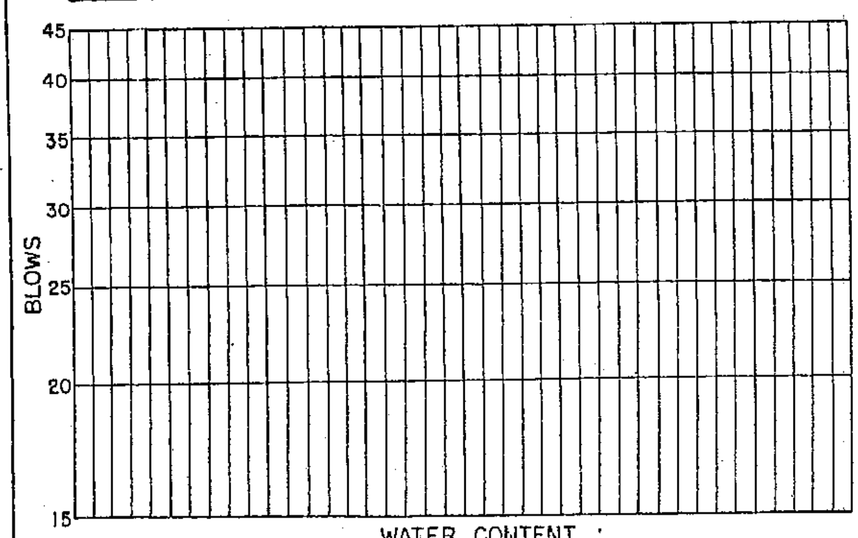
SIEVE ANALYSIS			% Passing		
-1 1/2"	126.29			312.80	
-1"	16.62	59.6	-40	12.32	4.9
-3/4	100.80	54.3	-80		
-4	26.68	22.1	-140		
-10	27.22	13.6	-200	2.87	0.9

GRAIN SIZE CURVE



LIQUID LIMIT DETERMINATION

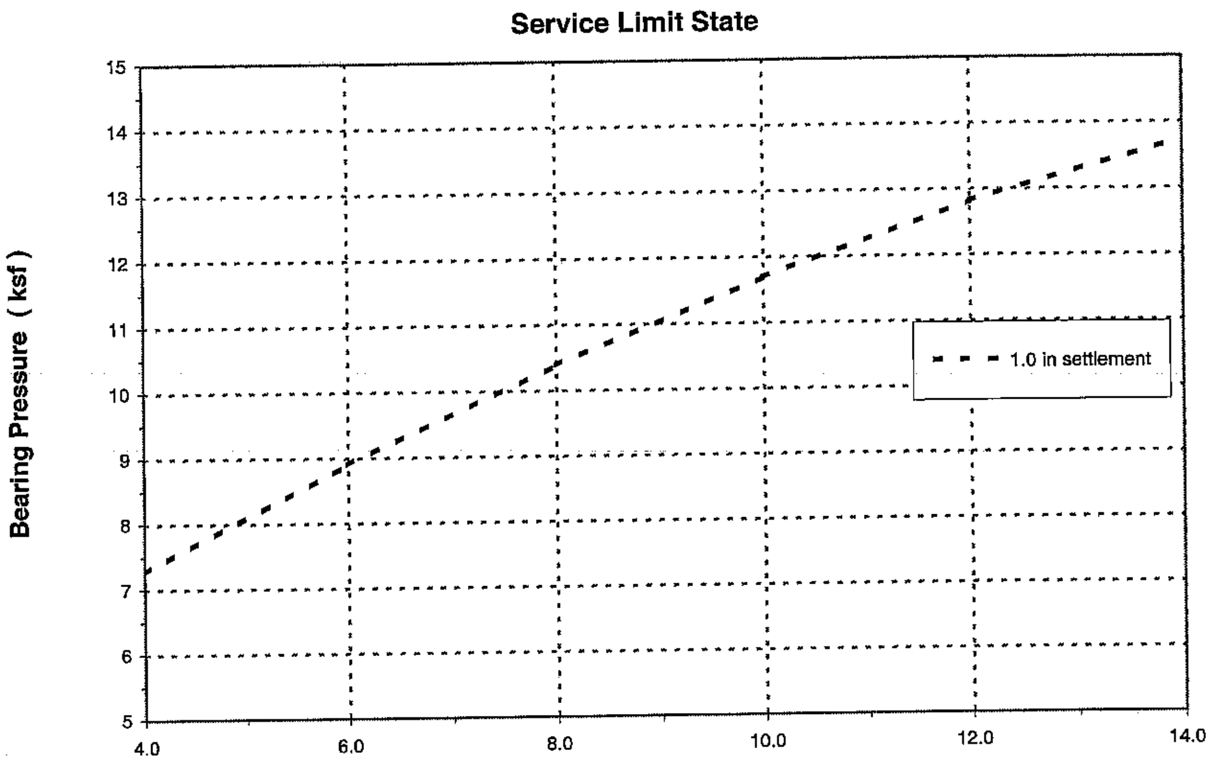
	LIQUID LIMIT				PLASTIC LIMIT
Can No.					
Wet Wt.					
Dry Wt.					
% H_2O					PL=
Blows					PI=



APPENDIX - D

Design Figures

Figure D-1	Design Curves for Spread Footing Option
Figure D-2	Drilled Shaft Capacity Chart – 2.5 ft diameter shaft
Figure D-3	Drilled Shaft Capacity Chart – 3.0 ft diameter shaft
Figure D-4	Drilled Shaft Capacity Chart – 3.5 ft diameter shaft
Figure D-5	Drilled Shaft Capacity Chart – 4.0 ft diameter shaft
Figure D-6	P-Y Curve Soil Data
Figure D-7	Typical MSE Wall Section
Figure D-8	Typical Modular Block Wall Detail



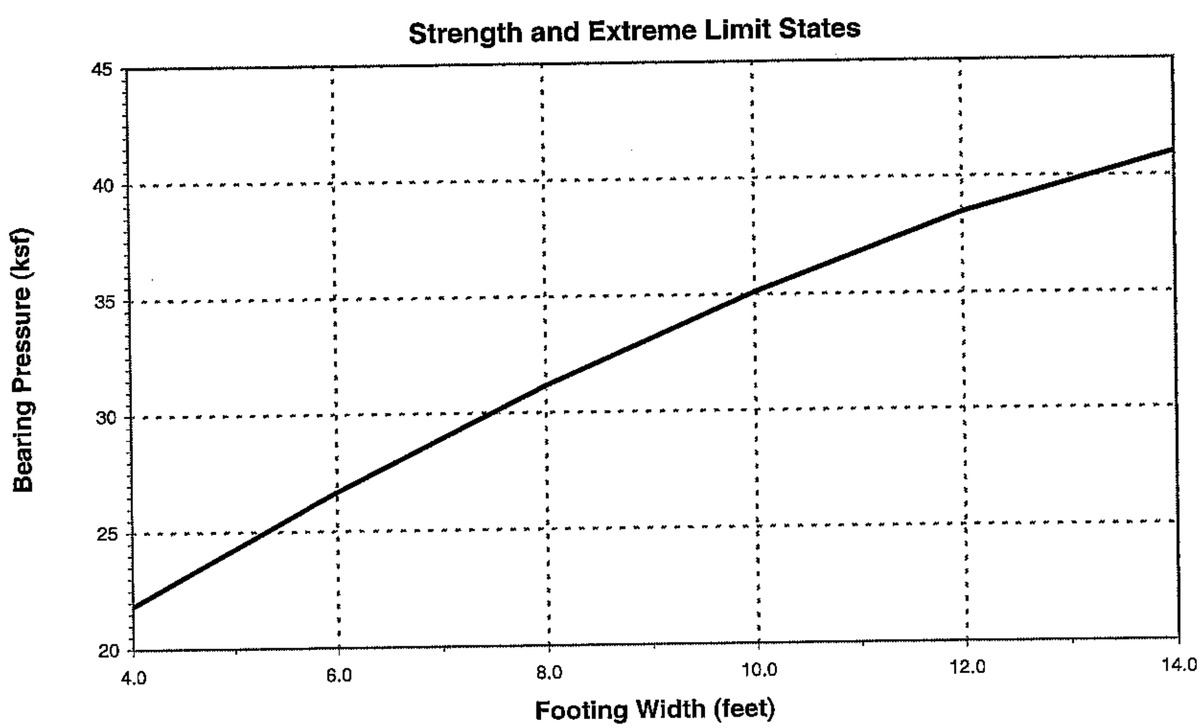


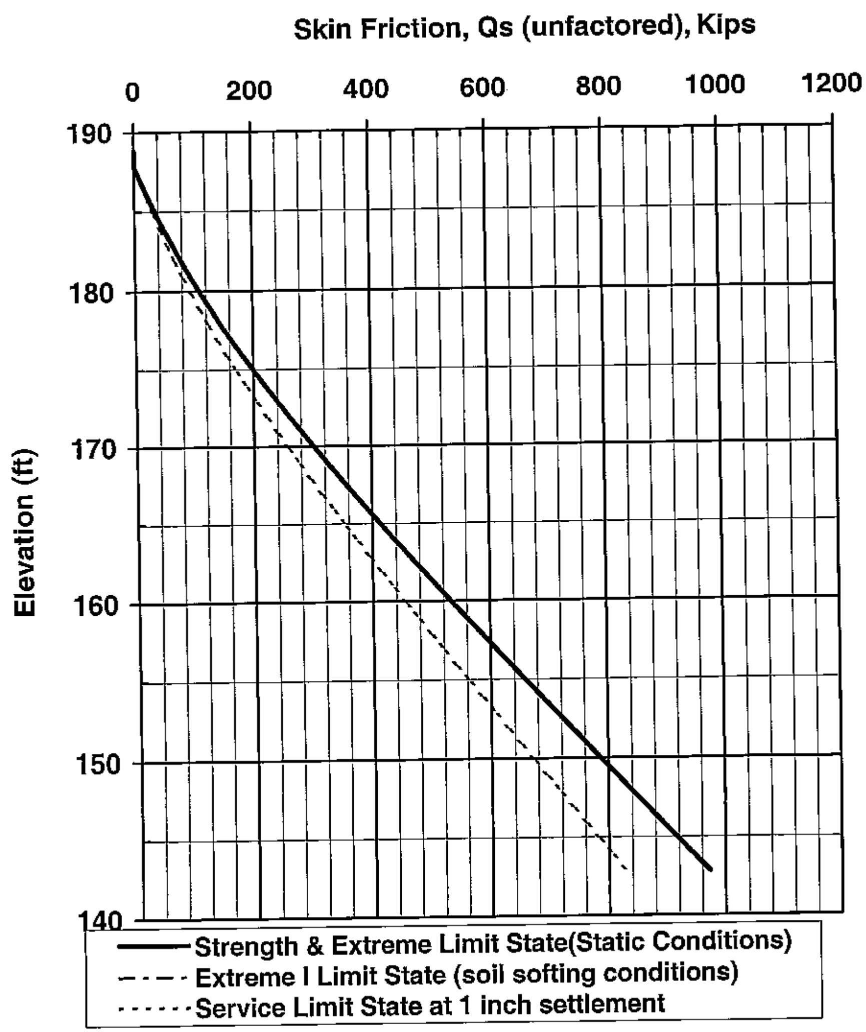
FIGURE D-1: Design Curves for Spread Footings at Piers 1, 2 and 3

Bottom of footing is based on minimum embedment criteria for bridge foundations.

SR-5 Chehalis Western Trails Pedestrain Bridge

Pier 2

Diameter = 2.5 feet
Shaft Type = Open Hole



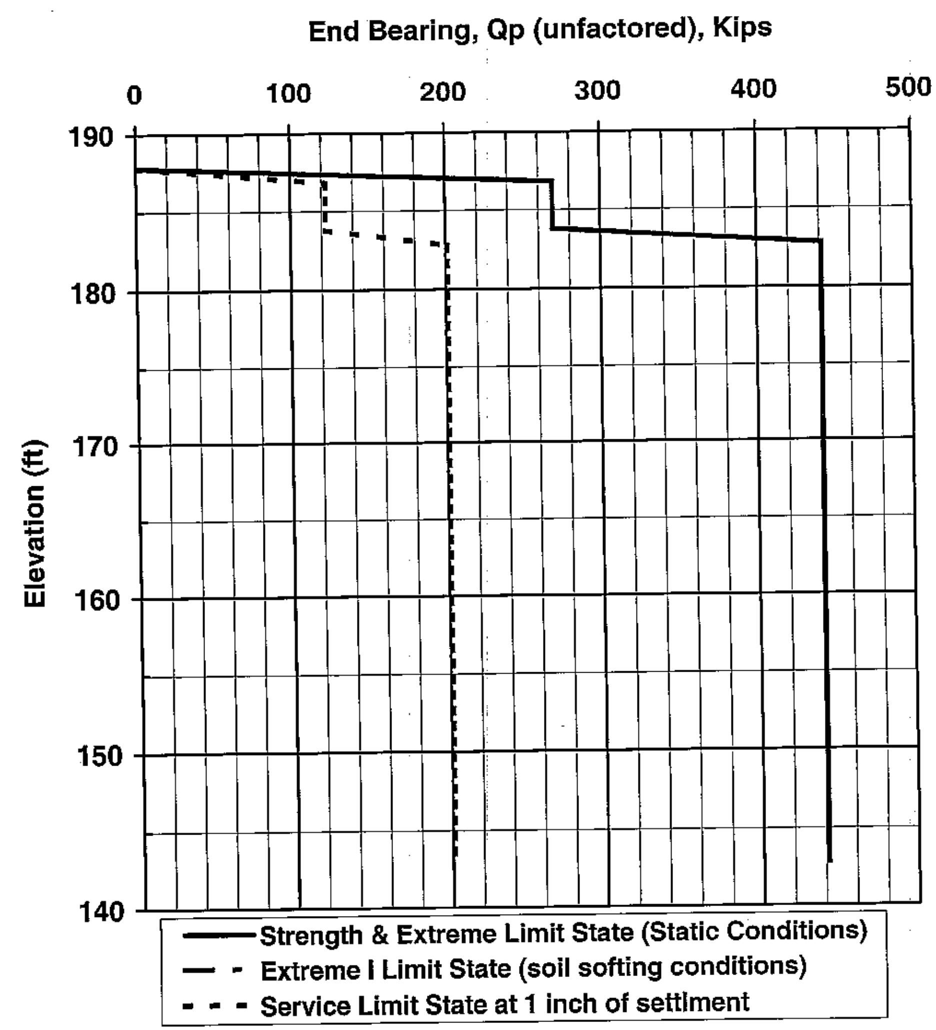
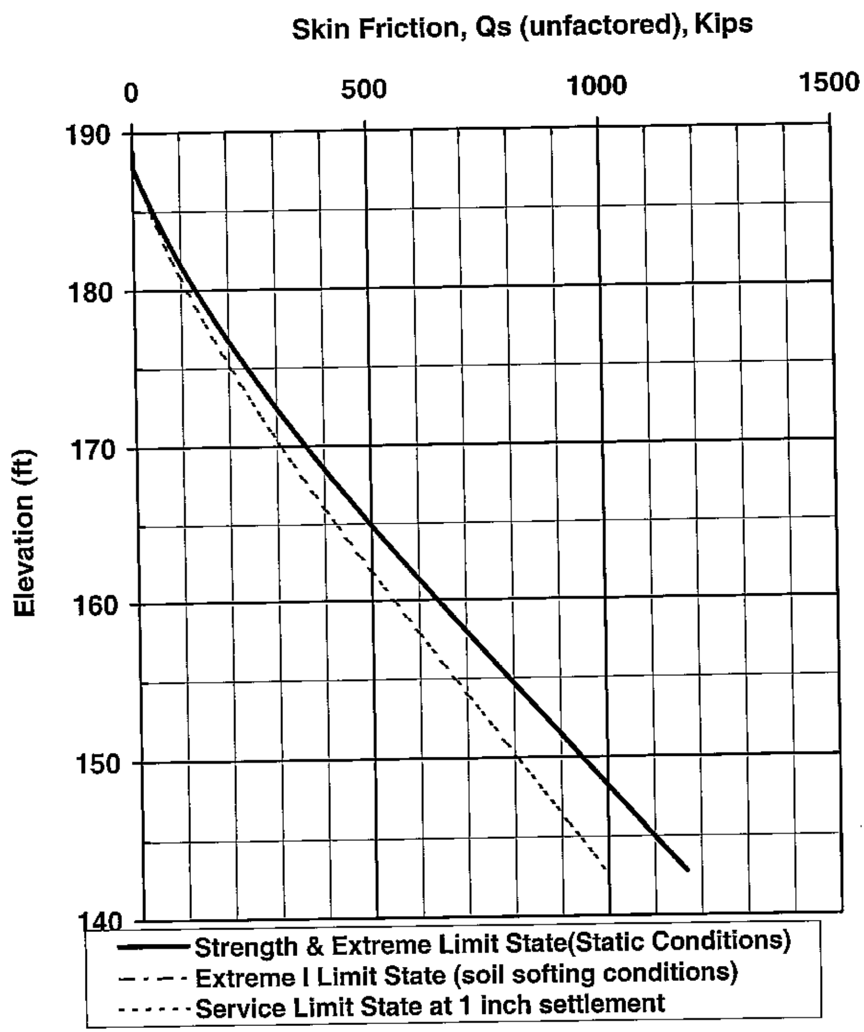


FIGURE D-2: Drilled Shaft Capacity Chart

SR-5 Chehalis Western Trails Pedestrain Bridge

Pier 2

Diameter = 3.0 feet
Shaft Type = Opern Hole



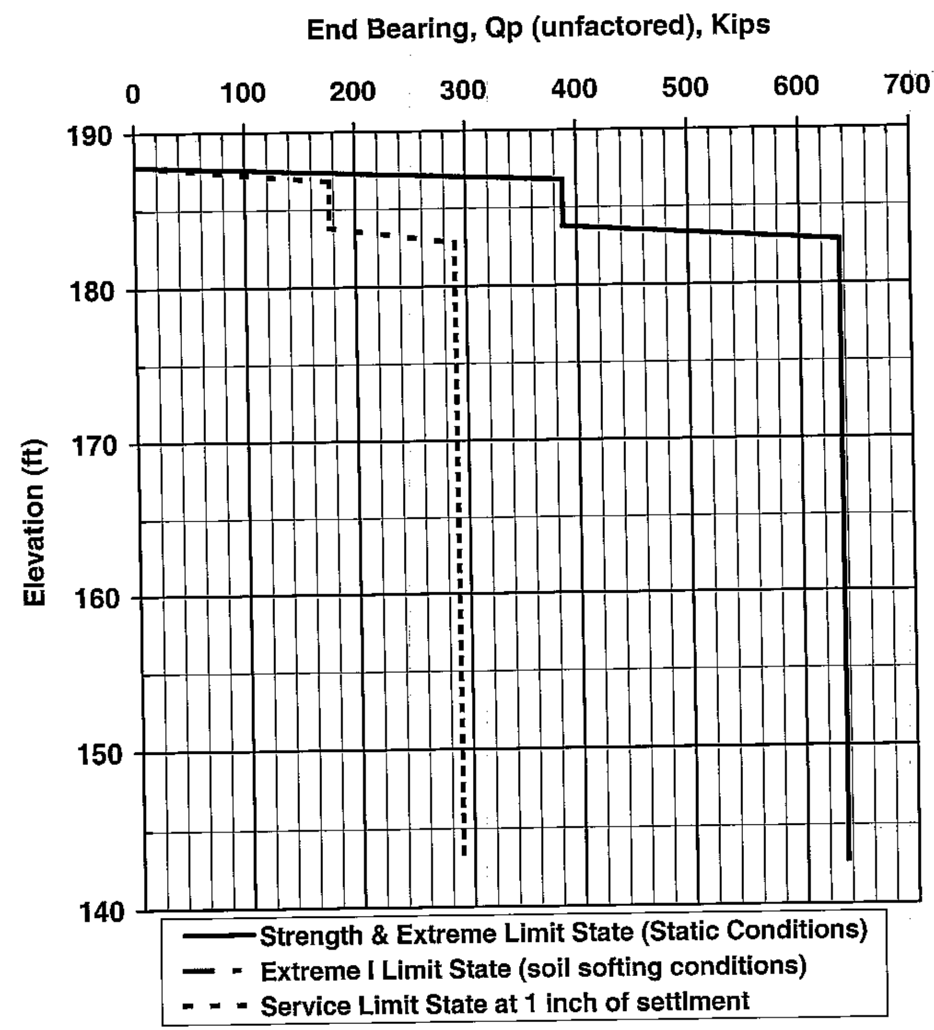
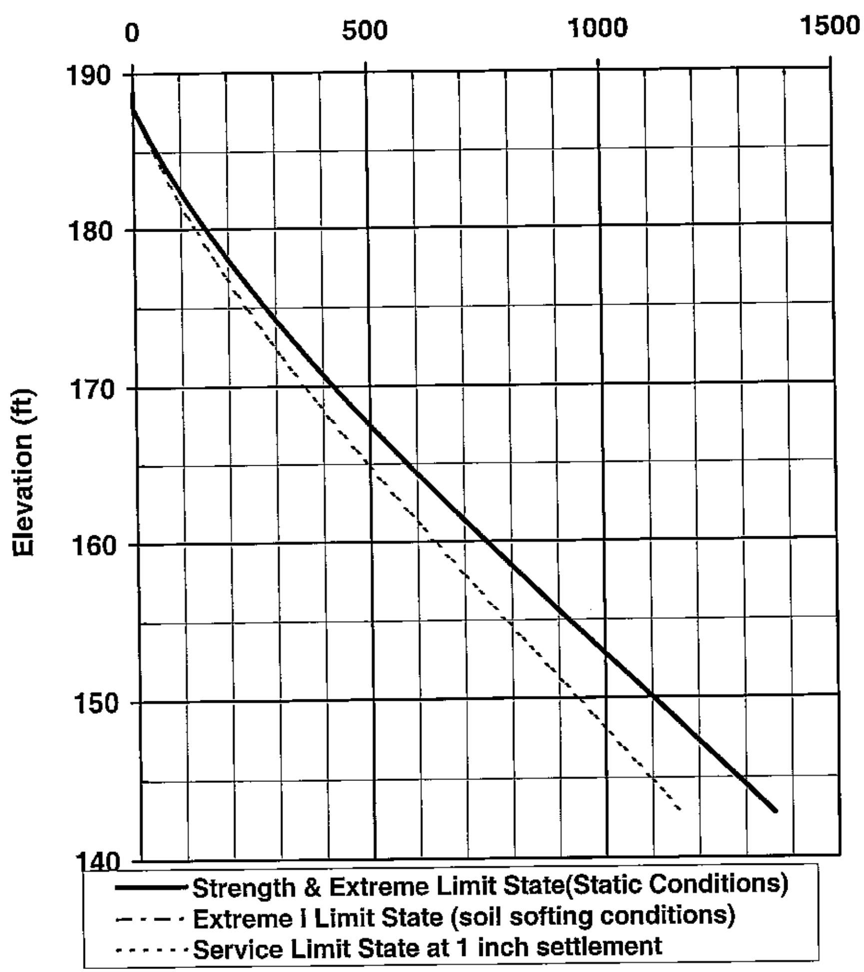


FIGURE D-3: Drilled Shaft Capacity Chart

SR-5 Chehalis Western Trails Pedestrain Bridge

Pier 2

Diameter = 3.5 feet
Shaft Type = Open Hole



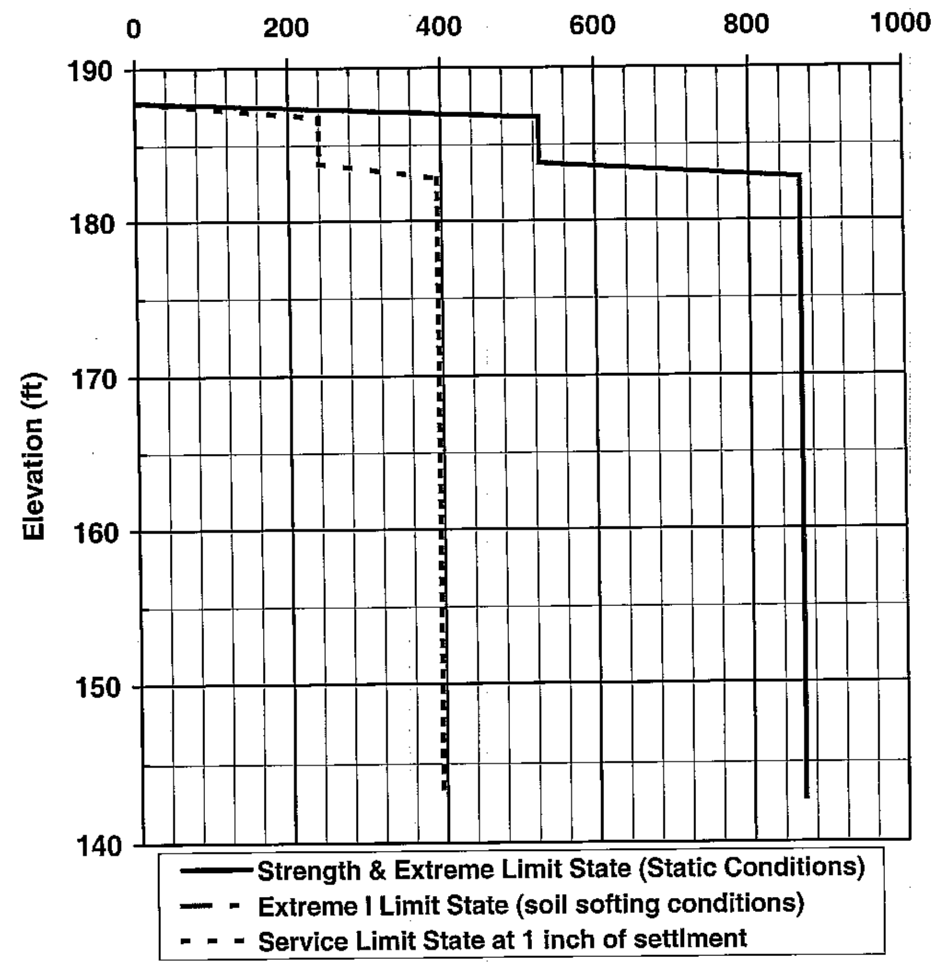
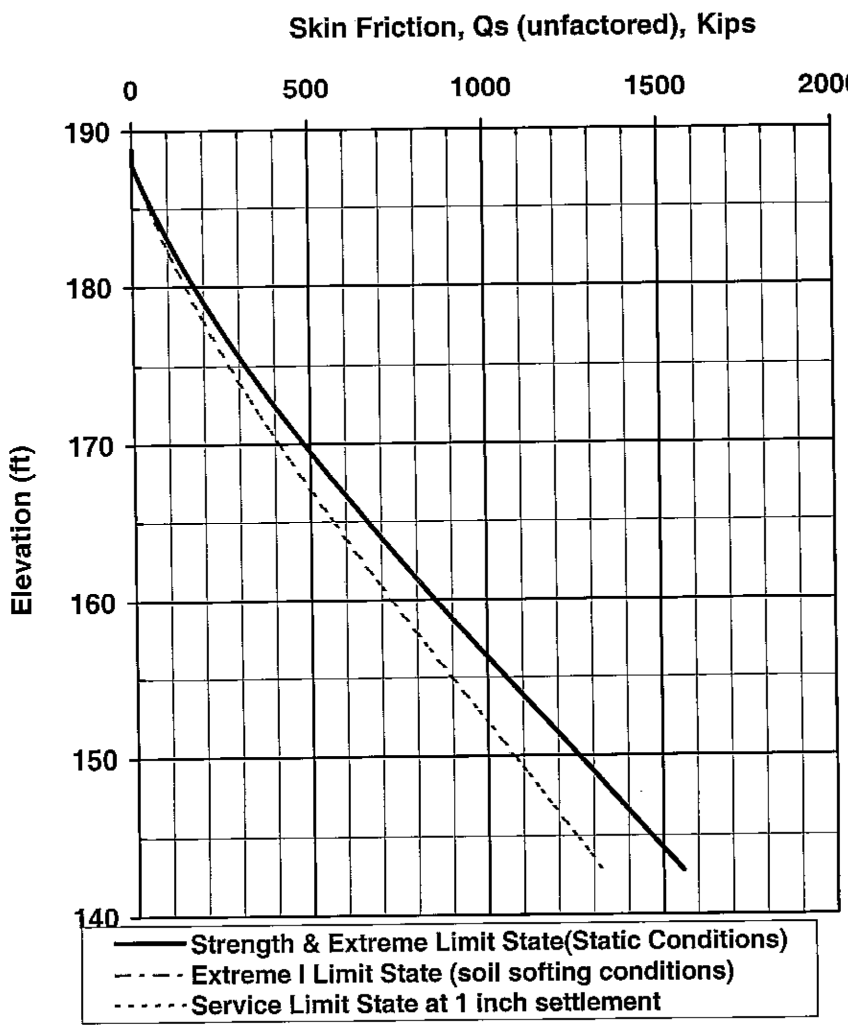


FIGURE D-4: Drilled Shaft Capacity Chart

SR-5 Chehalis Western Trails Pedestrain Bridge

Pier 2

Diameter = 4.0 feet
Shaft Type = Open Hole



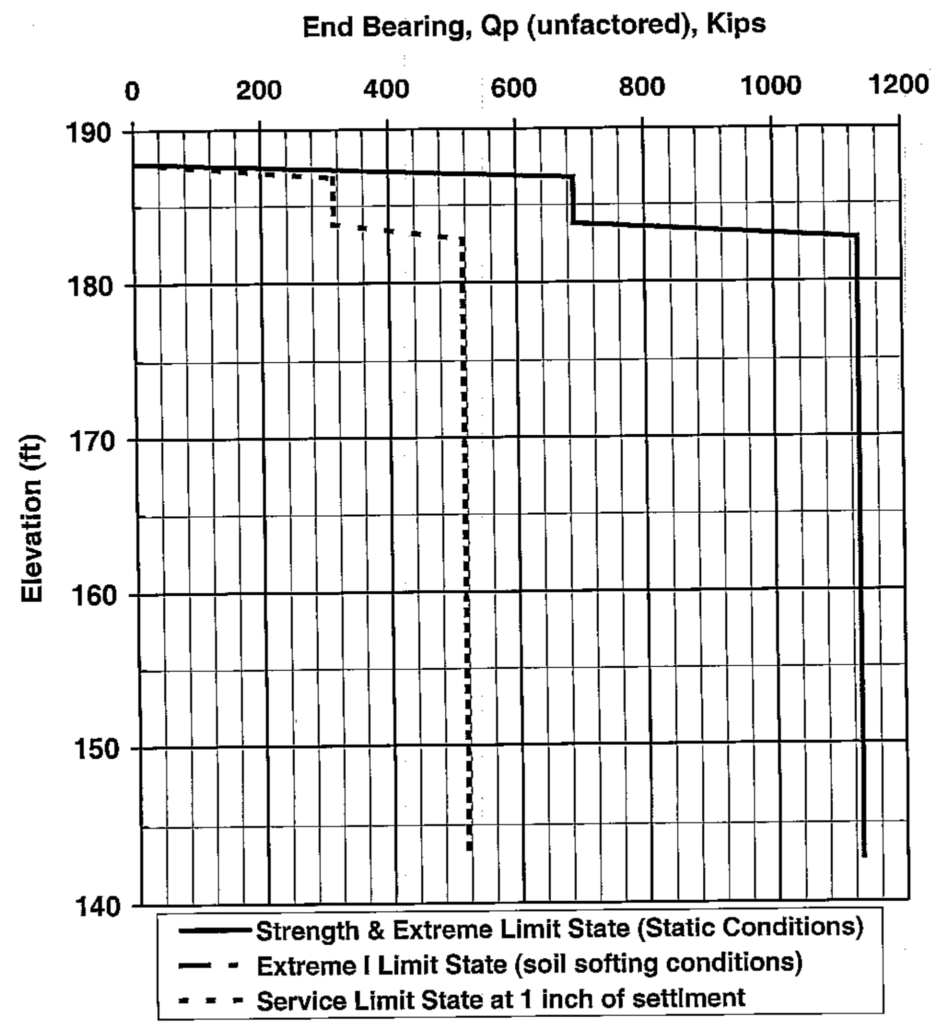


FIGURE D-5: Drilled Shaft Capacity Chart

SR-5 Chehalis Western Traisl Pedestrain Bridge
Pier 2 Drilled Shaft Station CWT 15+00.23

P-Y CURVE SOIL DATA for LPILE Program

Applies to : Pier 2

Ground Surface Elevation (ft): 196.8

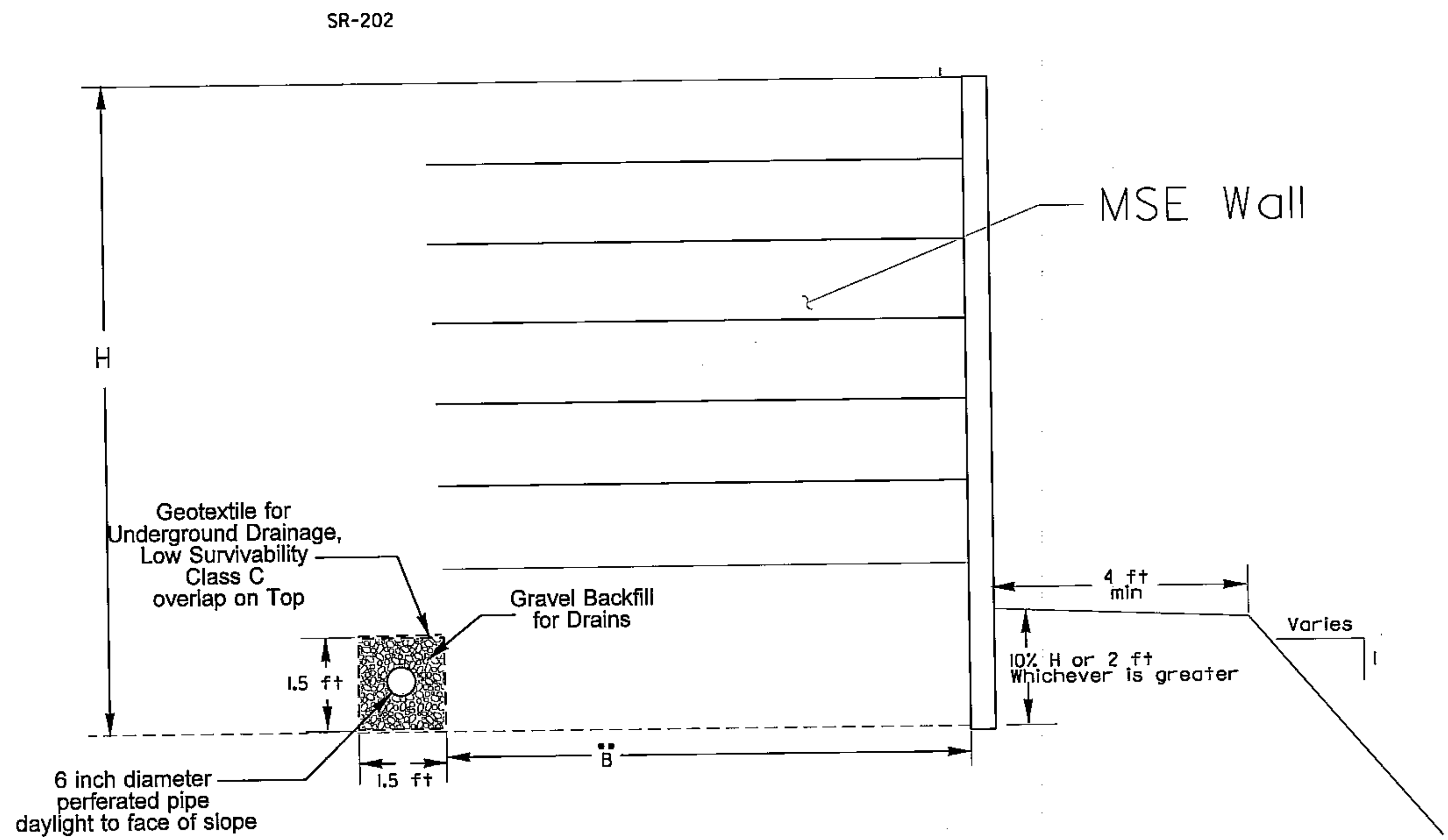
					STATIC ANALYSIS						
Soil Layer	Depth to Layer Bottom	Bottom of Layer Elevation	Soil Type	KSOIL	Effective Unit Weight of Soil		Cohesion		Axial Strain ε_{50}	Friction Angle ϕ	Modulus of Subgrade Reaction k
	ft	ft			lb/in3	pcf	psi	psf	(%)	(deg)	pci
1	5	192.0	Sand	4	0.072	125				35	135
2	8	189.0	Sand	4	0.036	62.6				35	85
3	14	183.0	Sand	4	0.039	67.6				38	120
4	60	137.0	Sand	4	0.042	72.6				45	200

Applies to : Pier 2

Ground Surface Elevation (ft): 196.8

					DYNAMIC ANALYSIS						
Soil Layer	Depth to Layer Bottom	Bottom of Layer Elevation	Soil Type	KSOIL	Effective Unit Weight of Soil		Cohesion		Axial Strain ε_{50}	Friction Angle ϕ	Modulus of Subgrade Reaction k
	ft	ft			lb/in3	pcf	psi	psf	(%)	(deg)	pci
1	5	192.0	Sand	4	0.072	125				35	135
2	8	189.0	Sand	4	0.036	62.6				35	85
3	14	183.0	Sand	4	0.039	67.6				38	120
4	60	137.0	Sand	4	0.042	72.6				45	200

FIGURE D-6 : P-Y CURVE SOIL DATA



JOB XL-2315 S.R. I-5 C.S. 3401 LAYOUT

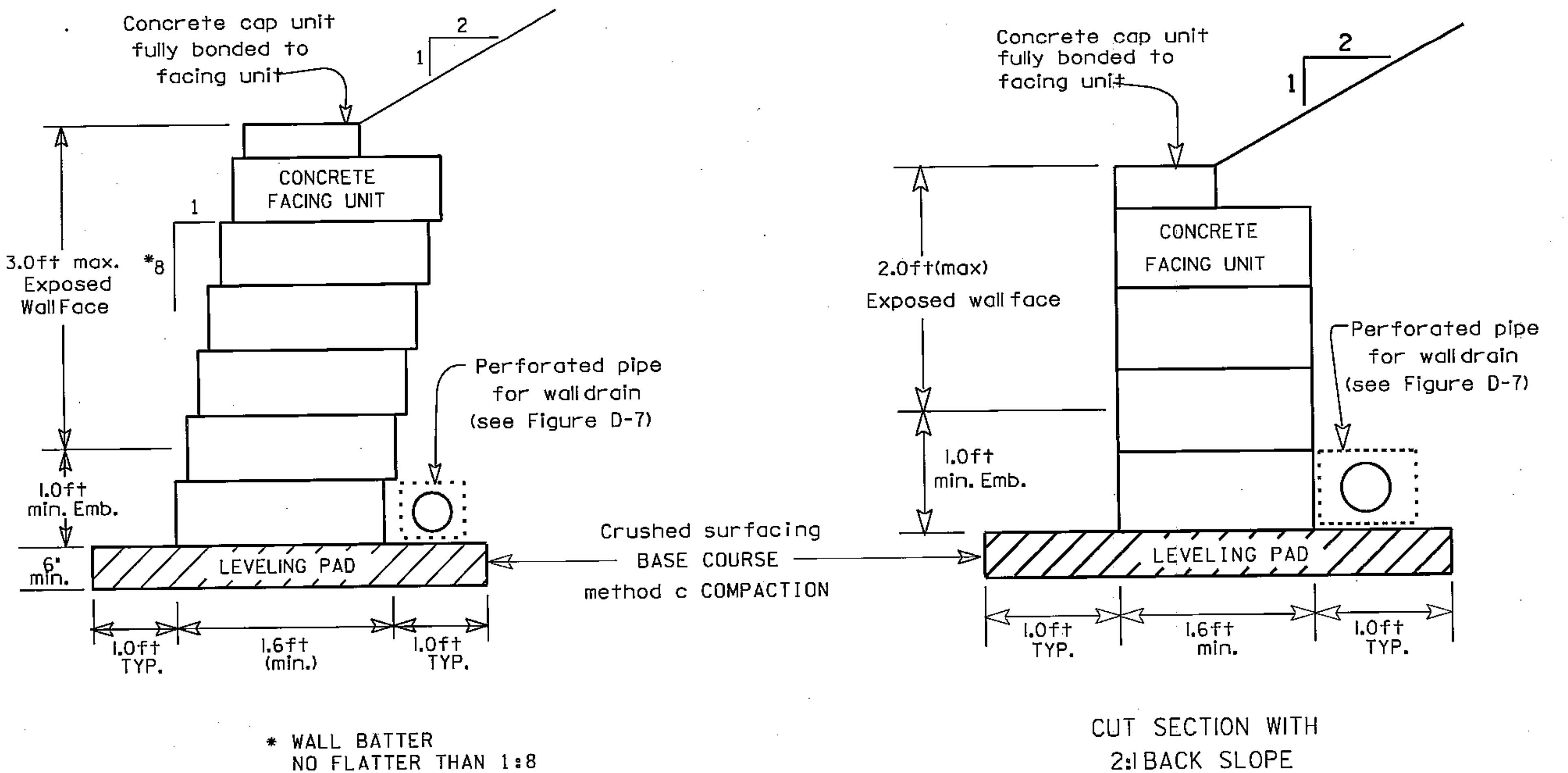
Chehalis Western Trail Pedestrian Bridge

WASHINGTON STATE TRANSPORTATION COMMISSION
DEPARTMENT OF TRANSPORTATION
MATERIALS BRANCH
T. E. BAKER MATERIALS ENGINEER

DATE 6/2005
SCALE Not to Scale VERT. HORIZ.
SHEET ___ OF ___
DRAWN BY WM

Figure D-4: Typical MSE Wall Section for Walls A, B, C, & D

...\WESTERN TRAIL BSD.dgn 08/03/2005 08:31:45 AM



JOB XL-2315 S.R. I-5 C.S. 3401 LAYOUT		
Chehalis Western Trail Pedestrian Bridge		
	WASHINGTON STATE TRANSPORTATION COMMISSION DEPARTMENT OF TRANSPORTATION MATERIALS BRANCH T. E. BAKER MATERIALS ENGINEER	DATE 6/2005 SCALE Not to Scale VERT. HORIZ. SHEET ___ OF ___ DRAWN BY DWG

Figure D-8: Typical Modular Block Wall Details
With no Reiforcement

APPENDIX - E

SPECIAL PROVISIONS

(Associated GSPs)

Structural Earth Walls

- Materials
- Submittals
- Construction Requirements

Geosynthetic Retaining Walls

- Materials

Shotcrete Facing

- Materials

Rock And Gravity Block Wall And Gabion Cribbing

- Materials
- Construction Requirements

Masonry Concrete Modular Modular Block Retaining Wall

SECTION 6-13, STRUCTURAL EARTH WALLS
April 5, 2004

(Associated GSPs)

STRUCTURAL EARTH WALLS

Materials
Section 6-13.2 is supplemented with the following:

(October 4, 2004)
Precast Concrete Panel Faced Structural Earth Wall Materials
General Materials

Concrete Leveling Pad
Leveling pad concrete shall be commercial concrete in accordance with Section 6-02.3(2)B.

Backfill for Precast Concrete Panel Faced Structural Earth Wall
All backfill material within the structural earth wall reinforced zone shall be free draining, free from organic or otherwise deleterious material.

Backfill material within the reinforced zone shall conform to Section 9-03.14(1), except that the maximum particle size for walls with geogrid reinforcement shall not exceed 1-1/4 inches.

All material within the structural earth wall reinforced zone shall be substantially free of shale or other soft, poor durability particles, and shall not contain recycled materials, such as glass, shredded tires, portland cement concrete rubble, or asphaltic concrete rubble. The material shall meet the following aggregate durability requirements:

Property	Test Method	Allowable Test Value
Los Angeles Wear, 500 rev.	AASHTO T 96	35 percent max.
Degradation	WSDOT Test Method 113	15 percent min.

For walls with metallic soil reinforcement, all material within the structural earth wall reinforced zone shall meet the following chemical requirements:

Property	Test Method	Allowable Test Value
Resistivity	AASHTO T 288	3,000 ohm-cm, min.
pH	AASHTO T 289	5 to 10
Chlorides	AASHTO T 291	100 ppm max.
Sulfates	AASHTO T 290	200 ppm max.

If the resistivity of the backfill material equals or exceeds 5,000 ohm-cm, the specified chloride and sulfate limits may be waived.

For walls with geogrid soil reinforcement, all material within the structural earth wall reinforced zone shall meet the following chemical requirements:

Property	Test Method	Allowable Test Value
pH	AASHTO T 289	4.5 to 9

Wall backfill material satisfying these gradation, durability, and chemical requirements shall be classified as nonagressive.

Proprietary Materials

ARES Modular Panel Wall System

Geogrid reinforcement shall conform to Section 9-33.1 and shall be the following products conforming to the specified material properties:

Geogrid Product Name	Wide Width Tensile Strength	[1,2]Long Term Tensile Strength, T_{al}
Tensar UX1600HS	8,980 lb./ft. min.	2,640 lb./ft.
Tensar UX1700HS	10,830 lb./ft. min.	3,280 lb./ft.

[1]These long term tensile strength requirements apply only in the geogrid direction perpendicular to the wall face.

[2]T_{al} shall be determined in accordance with WSDOT Standard Practice T 925, "Determination of Long-Term Strength for Geosynthetic Reinforcement".

The wide width tensile strength of the geogrid shall be a minimum average roll value (the average test results for any sampled roll in a lot shall meet or exceed the values shown in the table). The strength shall be determined in accordance with ASTM D 6637 for multi-rib specimens.

The ultraviolet (UV) radiation stability, ASTM D 4355, shall be a minimum of 70 percent strength retained after 400 hours in the weatherometer.

The longitudinal (i.e., in the direction of loading) and transverse (i.e., parallel to the wall or slope face) ribs that make up the geogrid shall be perpendicular to one another. The maximum deviation of the cross-rib from being perpendicular to the longitudinal rib (skew) shall be no more than 1 inch in 5 feet of geogrid width. The maximum deviation of the cross-rib at any point from a line perpendicular to the longitudinal ribs located at the cross-rib (bow) shall be 0.5 inches.

The geogrid shall not exhibit brittle fracture (snapping, or rapid crack development), when tested in accordance with Test Method WSDOT T 926.

The Engineer will take random samples of the geogrid materials at the job site. Approval of the geogrid materials will be based on testing of samples from each lot. A "lot" shall be defined as all geogrid rolls sent to the project site produced by the same manufacturer during a continuous period of production at the same manufacturing plant having the same product name. The Contracting Agency will require 14 calendar days maximum for testing the samples after their arrival at the WSDOT Materials Laboratory in Tumwater, WA.

The geogrid samples will be tested for conformance to the specified material properties. If the test results indicate that the geogrid lot does not meet the specified properties, the roll or rolls which were samples will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the specified properties, the entire lot will be rejected. If the test results from all the rolls retested meet the specified properties, the entire lot minus the roll(s) which failed will be accepted.

All geogrid materials which have defects, deterioration, or damage, as determined by the Engineer, will be rejected. All rejected geogrid materials shall be replaced at no expense to the Contracting Agency.

Except as otherwise noted, geogrid identification, storage and handling shall conform to the requirements specified in Section 2-12.2. The geogrid materials shall not be exposed to temperatures less than –20F and greater than 122F.

Rubber bearing pads shall be a type and grade as recommended by Tensar Earth Technologies, Inc.

Geosynthetic joint cover for all horizontal and vertical joints shall be a non-woven geosynthetic as recommended by Tensar Earth Technologies, Inc. Adhesive used to attach the geosynthetic to the rear of the precast concrete facing panel shall be as recommended by Tensar Earth Technologies, Inc.

MSE Plus Wall

Pins connecting the reinforcing mesh to the precast concrete panels shall conform to AASHTO M 32 and shall be galvanized in accordance with AASHTO M 111. Damage to the galvanizing shall be repaired with one coat of Formula A-9-73 paint conforming to Section 9-08.2.

Bearing pads shall be serrated high-density polyethylene (HDPE) copolymer pads with a Shore Hardness between 55 and 65.

Filter fabric joint cover for all horizontal and vertical joints shall be non-woven geosynthetic conforming to AASTHO M 288. Adhesive used to attach the geosynthetic to the rear of the precast concrete facing panel shall be as recommended by SSL, LLC.

Reinforced Earth Wall

Reinforcing strips shall be shop fabricated from hot rolled steel conforming to ASTM A 572 Grade 65 or approved equal, and shall be galvanized after fabrication in accordance with AASHTO M 111. Damage to the galvanizing shall be repaired with one coat of Formula A-9-73 paint conforming to Section 9-08.2.

Bolts and nuts shall conform to Section 9-06.5(3), and shall be galvanized in accordance with AASHTO M 232.

Rubber bearing pads shall be a type and grade as recommended by the Reinforced Earth Company.

Vertical joint filler between panels, when specified in the structural earth wall working drawings, shall be two inch square, flexible open cell polyether foam strips, Grade UU-34, as recommended by the Reinforced Earth Company.

Filter fabric joint cover for all horizontal and vertical joints, when specified in the structural earth wall working drawings, shall be a pervious woven polypropylene filter fabric as recommended by the Reinforced Earth Company. Adhesive used to attach the fabric material to the rear of the precast concrete facing panel shall be as recommended by the Reinforced Earth Company.

Reinforced Soil Wall
Reinforcing mesh shall be shop fabricated of cold drawn steel wire conforming to AASHTO M 32, and shall be welded into finished mesh fabric conforming to AASHTO M 55. Reinforcing mesh shall be galvanized after fabrication in accordance with AASHTO M 111. Damage to the galvanizing shall be repaired with one coat of Formula A-9-73 paint conforming to Section 9-08.2.

Retained Earth Wall
Tie strips shall be shop fabricated from hot rolled steel conforming to ASTM A 570 Grade 50 or approved equal, and shall be galvanized after fabrication in accordance with AASHTO M 111. Damage to the galvanizing shall be repaired with one coat of Formula A-9-73 paint conforming to Section 9-08.2.

The embed loops and connector bars shall be fabricated of steel wire conforming to AASHTO M 32, and shall be galvanized after fabrication in accordance with AASHTO M 111.

Filter fabric joint cover for all horizontal and inclined joints shall be a monofilament filter fabric as recommended by Foster Geotechnical. Adhesive used to attach the fabric to the rear of the precast concrete facing panel shall be as recommended by Foster Geotechnical.

Submittals
Section 6-13.3(2) is supplemented with the following:

(April 5, 2004)
The following geotechnical design parameters shall be used for the design of the structural earth wall(s):

Wall Name or No.: *** $$1$$ ***

Soil Properties	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight (pcf)	***$$2$$***	***$$3$$***	***$$4$$***
Friction Angle (deg)	***$$5$$***	***$$6$$***	***$$7$$***
Cohesion (psf)	***$$8$$***	***$$9$$***	***$$10$$***

	AASHTO Load Group I	AASHTO Load Group VII
Allowable Bearing Capacity (tsf)	***$$11$$***	***$$12$$***
Acceleration Coefficient (g)	N/A	***$$13$$***

Construction Requirements
Section 6-13.3 is supplemented with the following:

Precast Concrete Facing Panel and Concrete Block Fabrication
Section 6-13.3(4) is supplemented with the following:

(August 2, 2004)

Specific Fabrication Requirements for Proprietary Precast Concrete Panel Faced Structural Earth Walls

ARES Modular Panel Wall System

The concrete mix for precast concrete facing panels shall be a Contractor mix design in accordance with Section 6-02.3(2)A, producing a minimum compressive strength at 28 days of 4,500 psi. The Contractor mix design for precast concrete facing panels shall not include Type III cement unless otherwise approved by the Engineer.

The slot opening for geogrid attachment in precast concrete facing panels shall be 1/8 inch minimum. The Contractor shall test the slot opening of each concrete panel using a feeler gauge furnished by Tensar Earth Technologies, Inc. Concrete panels with slot dimension deviations that allow the feeler gauge to be pulled out of the slot shall be rejected.

MSE Plus Wall

The concrete mix for precast concrete facing panels using soil reinforcement mesh of either 6w20 or 6w24 shall be a Contractor mix design in accordance with Section 6-02.3(2)A, producing a minimum compressive strength at 28 days of 5,000 psi. The Contractor mix design for all precast concrete facing panels shall include Type II cement only.

Rods forming the internal connection channel in precast concrete facing panels shall be turned within 20 minutes of concrete placement in each concrete panel, and removed between 3 and 24 hours after concrete placement.

Precast Concrete Facing Panel and Concrete Block Erection

Section 6-13.3(5) is supplemented with the following:

(April 5, 2004)

Specific Erection Requirements for Proprietary Precast Concrete Panel Faced Structural Earth Walls

MSE Plus Wall

The loop pockets and access pockets of the internal connection channel of the precast concrete facing panels shall be cleaned of all backfill and extraneous materials prior to inserting the pins to connect the soil reinforcing mesh to each concrete panel.

(April 5, 2004)

Specific Erection Requirements for Proprietary Concrete Block Faced Structural Earth Walls

Mesa Wall

For all concrete block courses receiving geogrid reinforcement, the fingers of the block connectors shall engage the geogrid reinforcement apertures, both in the connector slot in the block, and across the block core. For all concrete block courses with intermittent geogrid coverage, a #3 steel reinforcing bar shall be placed, butt end to butt end, in the top block groove, with the butt ends being placed at a center of a concrete block.

Backfill

Section 6-13.3(7) is supplemented with the following:

(April 5, 2004)
Specific Backfill Requirements for Proprietary Precast Concrete Panel Faced Structural Earth Walls

MSE Plus Wall

At each wall reinforcement level, the Contractor shall place the backfill to the level of the connection. Backfill placement and compaction methods shall ensure that no voids exist directly beneath the wall reinforcement near the precast concrete facing panels.

(August 2, 2004)
Precast Concrete Panel Faced Structural Earth Wall

Precast concrete panel faced structural earth walls shall be constructed of only one of the following wall systems. The Contractor shall make arrangements to purchase the precast concrete panels, soil reinforcement, attachment devices, joint filler, and all necessary incidentals from the source identified with each wall system:

ARES Modular Panel Wall System

ARES Modular Panel Wall System is a registered trademark of Tensar Earth Technologies, Inc.

Tensar Earth Technologies, Inc.
5883 Glenridge Drive Suite 200
Atlanta, GA 30328
(800) 836-7271

MSE Plus Wall

MSE Plus is a registered trademark of SSL, LLC.

SSL, LLC
4740-E Scotts Valley Drive
Scotts Valley, CA 95066
(831) 430-9300
FAX (831) 430-9340

Reinforced Earth Wall

Reinforced Earth is a registered trademark of the Reinforced Earth Company.

The Reinforced Earth Company
20381 Lake Forest Drive Suite B-2
Lake Forest CA, 92630
(949) 587-3060

Reinforced Soil Wall

Reinforced Soil is a registered trademark of Hilfiker Retaining Walls.

Hilfiker Retaining Walls
P. O. Box 2012
Eureka, CA 95501-2012
(707) 443-5093
FAX (707) 443-2891

Retained Earth Wall

Retained Earth is a registered trademark of Foster Geotechnical.

Foster Geotechnical
1660 Hotel Circle North Suite 304
San Diego, CA 92108
(619) 688-2400
FAX (619) 688-2499

(April 5, 2004)
Geosynthetic Properties for Permanent Geosynthetic Retaining Walls

Table 9: Long-term tensile strength, T_{al}, required for the geosynthetic reinforcement used in geosynthetic retaining walls.

[3]Wall Location	Vertical Spacing of Reinforcement Layers	Reinforcement Layer Distance from Top of Wall	[1,2,3]Minimum Long-Term Tensile Strength, T_{al}
*** $$1$$ ***	*** $$2$$ ***	*** $$3$$ ***	*** $$4$$ ***

[1]These long-term tensile strength requirements apply only in the geosynthetic direction perpendicular to the wall face.

[2]T_{al} shall be determined in accordance with WSDOT Test Method 925.

[3]Walls *** $$5$$ *** are classified as Class *** $$6$$ *** structures.

SECTION 6-18, SHOTCRETE FACING
April 5, 2004

(Associated GSPs)

SHOTCRETE FACING

Materials

Section 6-18.2 is supplemented with the following:

(August 2, 2004)
Shotcrete Facing
Portland cement shall be Type I or II in accordance with Section 9-01.2(1).

Air entrainment shall be 6.0 percent, $\pm$ 1.5 percent.

Water for mixing and curing shall be clean and free from substances, which may be injurious to concrete or steel, and shall be free of elements which would cause staining.

Aggregate for shotcrete shall meet the following gradation requirements:

Sieve Size	Percent Passing by Weight
1/2 inch	100
3/8 inch	90 to 100
U.S. No. 4	70 to 85
U.S. No. 8	50 to 70
U.S. No. 16	35 to 55
U.S. No. 30	20 to 35
U.S. No. 50	8 to 20
U.S. No. 100	2 to 10
U.S. No. 200	0 to 2.5

MASONRY CONCRETE MODULAR BLOCK RETAINING WALL

Description

Where shown in the Plans or where designated by the Engineer, the Contractor shall construct masonry concrete modular block retaining wall.

Quality Assurance

The completed walls shall meet the following tolerances:

1. Deviation from the design batter and horizontal alignment shall not exceed one (1) inch when measured along a 10 ft straight edge. The face batter deviation measurement will be made at the midpoint of each facing block layer

2. Deviation from the overall design batter of the wall shall not exceed 0.75 inches per 10 ft of wall height

3. The base of the retaining wall excavation shall be within 3 inches of the staked elevations unless otherwise directed by the Engineer

4. The external wall dimensions shall be placed within 2 inches of that staked on the ground.

5. The wall manufacturer shall provide a qualified and experienced representative at the job site at the start of wall construction and as needed to resolve construction problems. Recommendations made by the representative and approved by the Engineer shall be followed by the Contractor.

Submittals

The Contractor shall, a minimum of 30 days prior to beginning wall construction, submit to the Engineer for approval, a construction manual that provides step-by-step directions for construction of the wall system. The manual shall include, but not be limited to the following:

1. Detailed Plans for each wall.

2. Proposed wall construction method, including types of equipment to be used and proposed erection sequence.

3. Certificate of compliance, samples, and test data for the purpose of acceptance.

Materials

General

Material specifications in these Special Provisions shall take precedence over requirements of the wall manufacturer, unless otherwise approved by the Engineer in advance of their use in the construction.

Backfill Material

Backfill material for the wall shall meet the requirements of Section 9-03.12(2), Gravel backfill for Walls.

Backfill For Leveling Pad

Material for the wall face leveling pad shall meet the requirements of Section 9-03.9(3), Crushed Surfacing Base Course.

Concrete Facing Units

The concrete facing units shall be constructed to the dimension and shape as detailed in the Plans and shall meet all the materials, manufacturing, and physical requirements of ASTM C1372 except for the following:

Compressive Strength minimum, per ASTM C-140	4,000 psi at 28 days
Water absorption maximum	5 percent

In lieu of the water absorption requirement, the wall or block manufacture shall provide freeze thaw test data, specific to the block supplier used, conducted in accordance with ASTM C1262, with acceptance as the weight loss of each 4 of the 5 specimens at the conclusion of 150 freeze-thaw cycle does not exceed 1% of its initial weight.

The block manufacturer shall provide a certificate of compliance and test data for the lot of material sent to the job site demonstrating that the property requirements provided above have been met. The maximum lot size for the purposes of this specification shall be 2,000 blocks. Testing and inspection of the dry cast concrete blocks shall conform to ASTM C140. Acceptability of the panels and blocks will be determined on the basis of compressive strength test and visual inspection.

The units shall have angled sides and capable of attaining concave and convex alignment curves with a minimum radius of 3.5 feet. The facing units shall have a broken face finish. The height of each individual block shall be within ± 1/16 inches of the specified dimension. The length and width of each block shall be within ± 1/8 inches of the specified dimensions. Hollow units shall have a minimum wall thickness of 1.25 inches. Any blocks, which have defects, which indicate imperfect molding, defects indicating honeycomb or open texture concrete, or cracked or severely chipped blocks will be rejected.

Facing units shall be interlocked with concrete shear keys or non-corrosive polyester/fiberglass or polyethylene solid pins. The pins shall have a minimum diameter of 1/2 inch.

Foundation

Excavation shall be in accordance with the requirements of Section 2-09 and in close conformance to the limits shown in the Plans.

The foundation subgrade for the structure shall be graded level and compacted to the satisfaction of the Engineer. Once foundation construction has begun, the Contractor shall take precaution to direct surface runoff from adjacent areas away from the wall construction site.

The wall face leveling pad shall be compacted using Method C. The surface of the leveling pad shall be hard and level. The leveling pad shall be prepared in a manner, which will insure complete contact between each wall facing unit in the lowest course of facing units and the leveling pad.

Wall Erection

The walls shall be constructed to the configurations, lines and grades shown in the Plans and in accordance with the Manufacturer's recommendations as approved by the Engineer.

Wall construction shall begin at the lowest portion of the excavation and each layer shall be placed horizontally. The first course of the wall units shall be checked for level and proper alignment, prior to proceeding with the next course.

Facing units may be saw cut as required using standard masonry tools. Sawn, half width blocks shall not be used in the base course.

If connecting pins are used between facing units for shear restraint, they shall be installed in each concrete facing unit in a manner, which will insure that the pins will protrude a minimum of one (1) inch into each facing unit.

All voids within and around the concrete facing units shall be filled with Gravel Backfill for Walls and tamped in place. Excess fill material shall be swept from the surface of the concrete facing blocks.

Wall facing units shall be turned into the embankment at the ends of each course where the change in the wall elevation is greater than the individual block height. A minimum of three units shall be installed below grade at these ends. End returns are not required for elevation changes of which are less than the block height.

The cap course shall be bonded to the adjacent lower course with an approved cement base, waterproof anchoring cement.

Backfill Placement
Each layer of backfill shall be compacted to 95 percent of maximum density. The water content of the wall backfill shall not deviate above the optimum water content by more than 3 percent. Compaction within 3 feet of the wall face shall be achieved using light mechanical tampers approved by the Engineer and shall be done in a manner to cause no damage or distortion to the wall facing elements.

Measurement
The masonry retaining wall will be measured by the square foot of completed wall in place. The vertical limits for measurement are from the foundation to the top of the wall. The horizontal limits for measurement are from end of wall to the end of wall.

Gravel backfill for walls will be measured by the cubic foot in place determined by the limits shown in the plans.

Structure excavation will be measured by the cubic yard to the limits shown in the Plans.

Payment
The unit contract price per square meter for "Retaining Wall" shall be full pay for performing the wall construction work.

The Unit contract price per cubic meter for "Gravel backfill for Walls Incl. Haul" shall be full pay for furnishing, processing, hauling, placing, and compacting the backfill material.